Santa Cruz Wharf

Engineering Report ATTACHMENTS

Prepared for:

City of Santa Cruz

Prepared by:

moffatt & nichol

2185 North California Blvd, Suite 500

Walnut Creek, CA 94596

October 8, 2014

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Attachment 1A Photographs

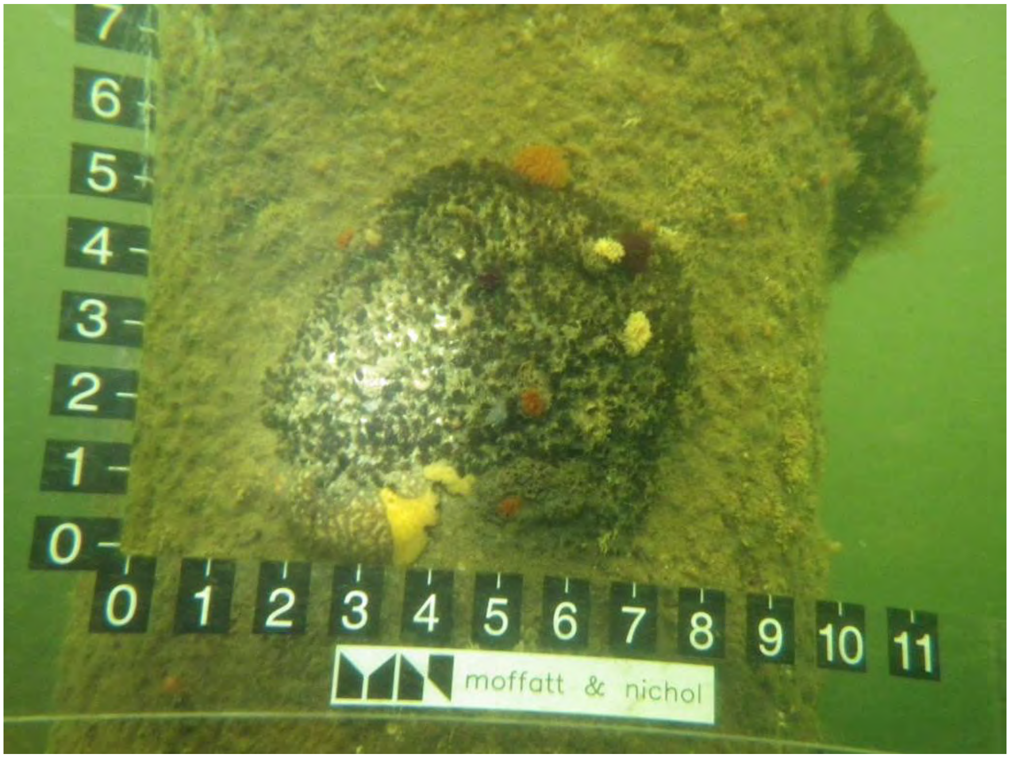

Photo 1-8: Bent 41 pile in excellent condition

Photo 1-9: Bent 76, Pile 14 – Portion of pile with no damage

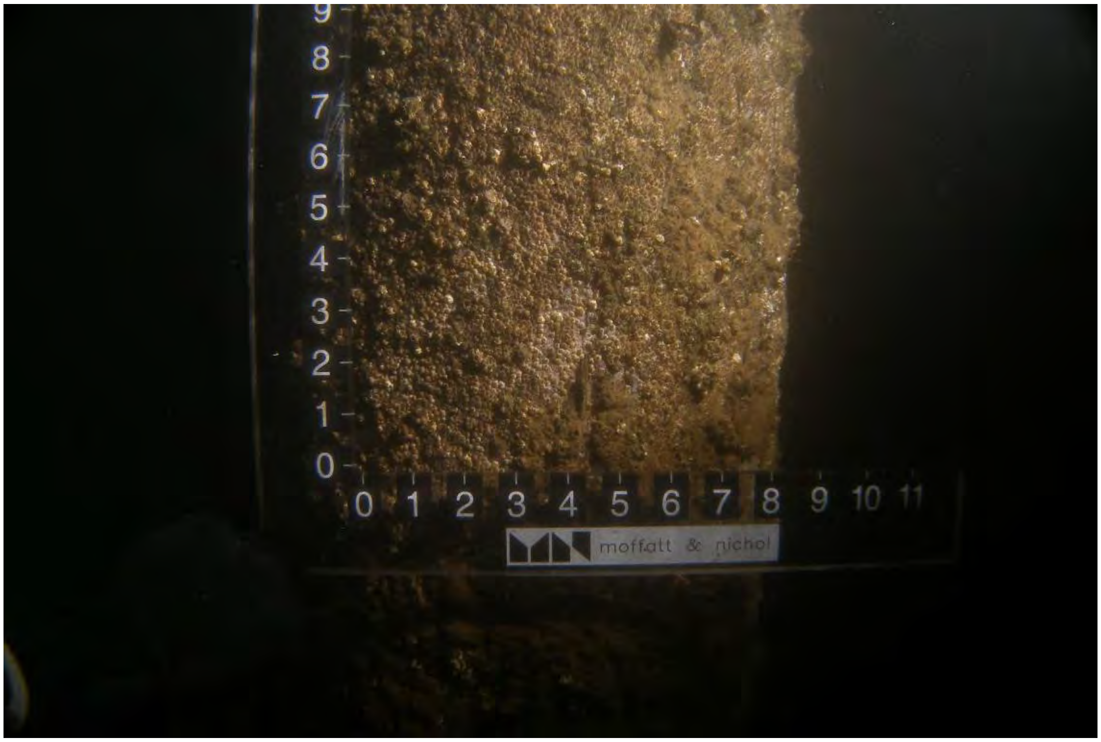

Photo 1-10: Bent 76, Pile North of 20, portion of pile in excellent condition

Photo 1-11: Bent 78, Pile South of 20, excellent condition

Photo 1-12: Bent 112, Pile 5, portion with excellent condition

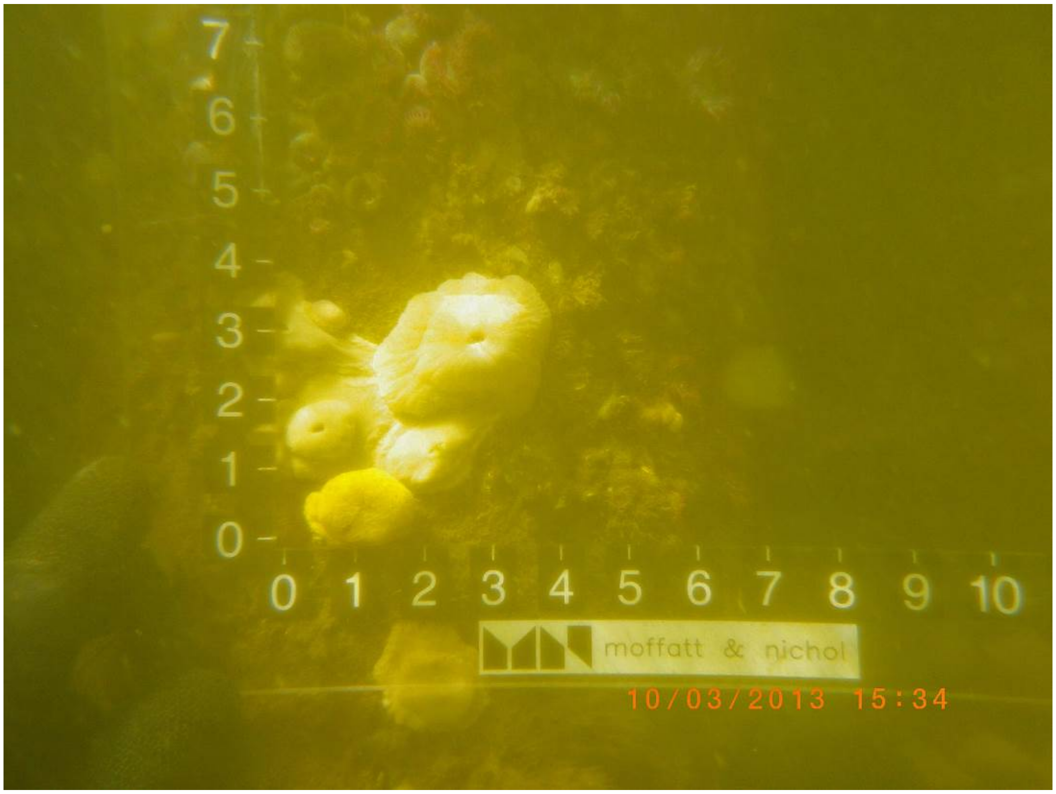

Photo 1-13: Bent 154, Pile in excellent condition

Photo 1-14: Bent 154, Pile in excellent condition

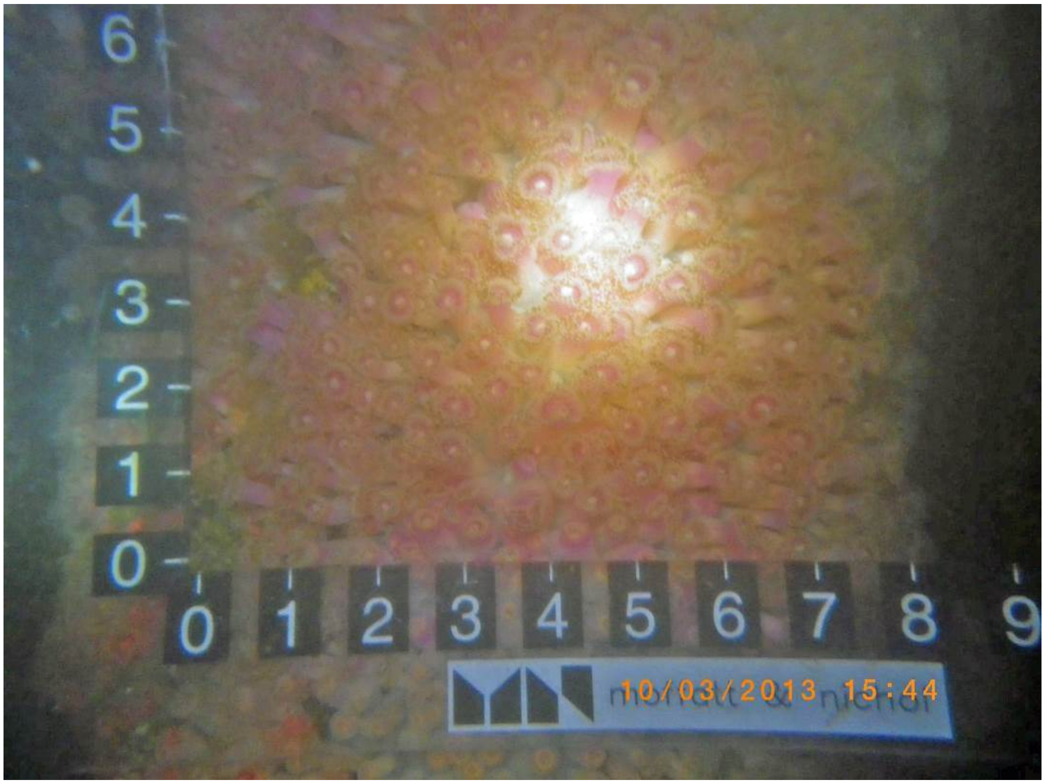

Photo 1-15: Bent 155, Pile in excellent condition

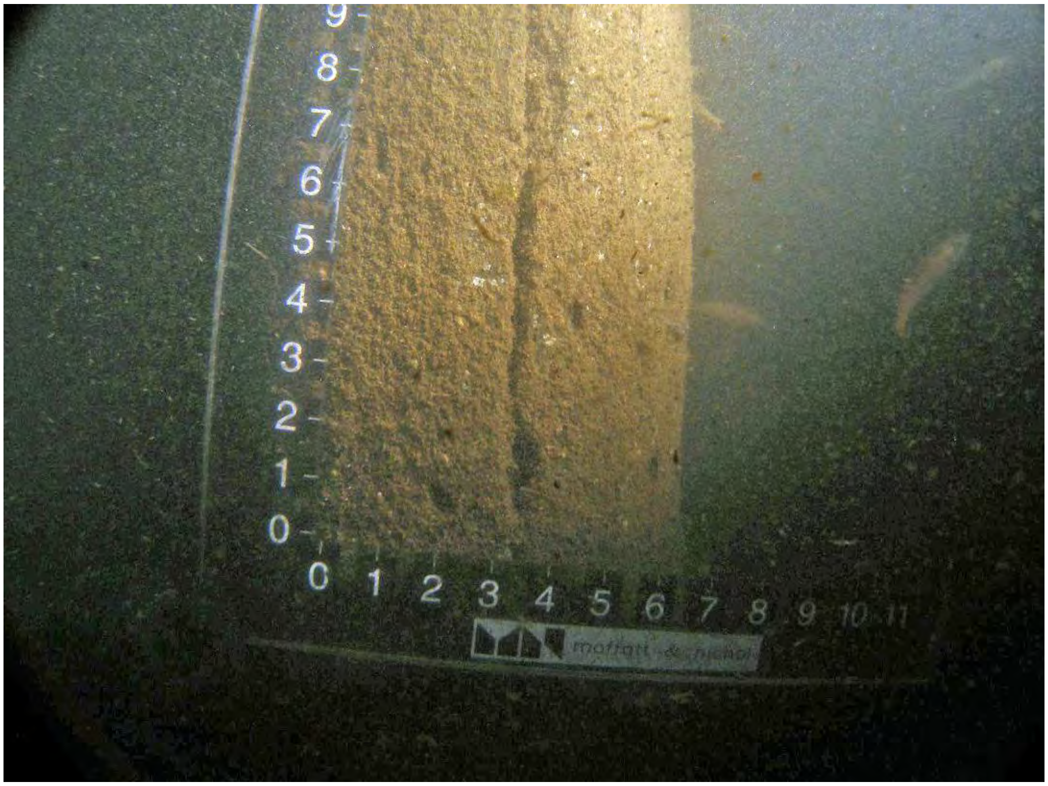

Photo 1-16: Bent 75, Pile 14 – Crack with 5% section loss inside

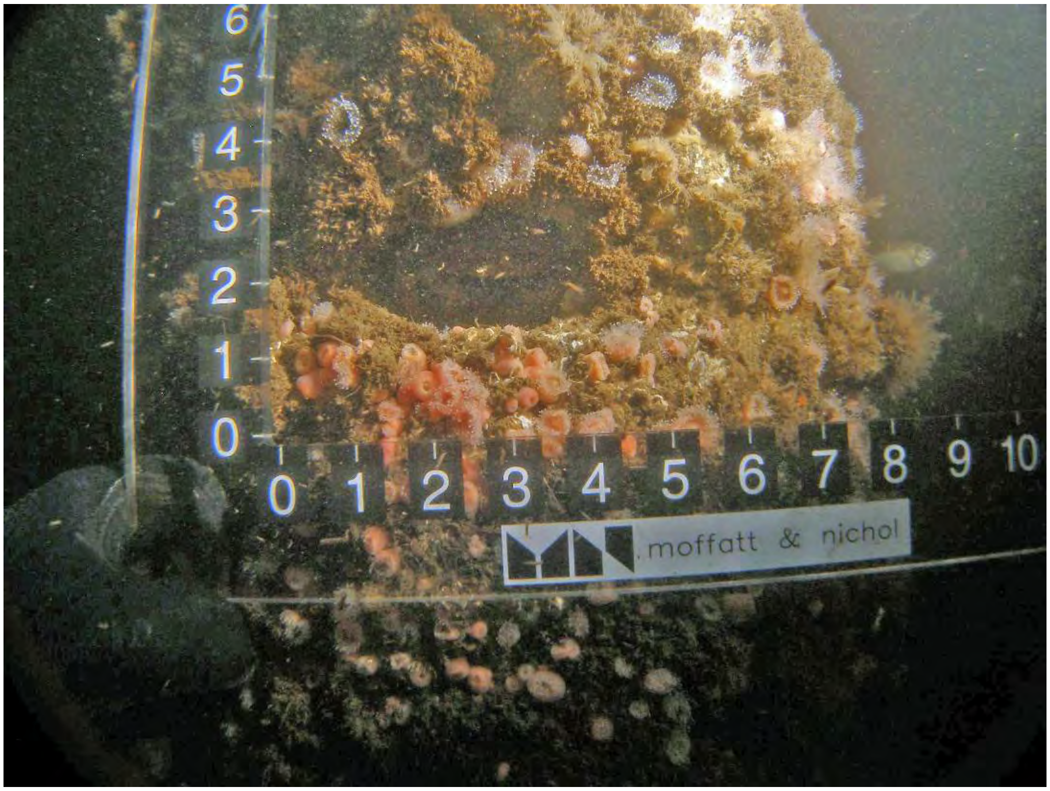

Photo 1-17: Bent 76, Pile North of 20, portion of pile 5% section loss

Photo 1-18: Bent 112, Pile 5 - Portion with splits

Photo 1-19: Bent 75, Pile 4 – Crack with 50% section loss inside

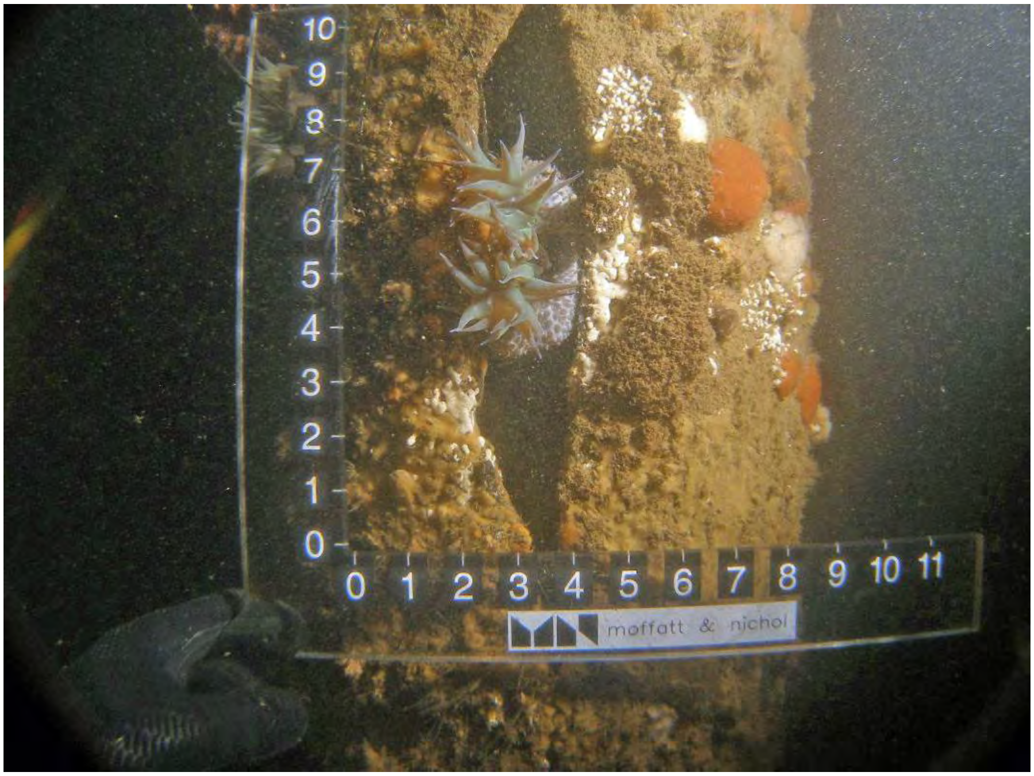

Photo 1-20: Bent 75, Pile 6 – Crack with 60% section loss inside

Photo 1-21: Bent 112, Pile 7, Hole with 70% section loss inside

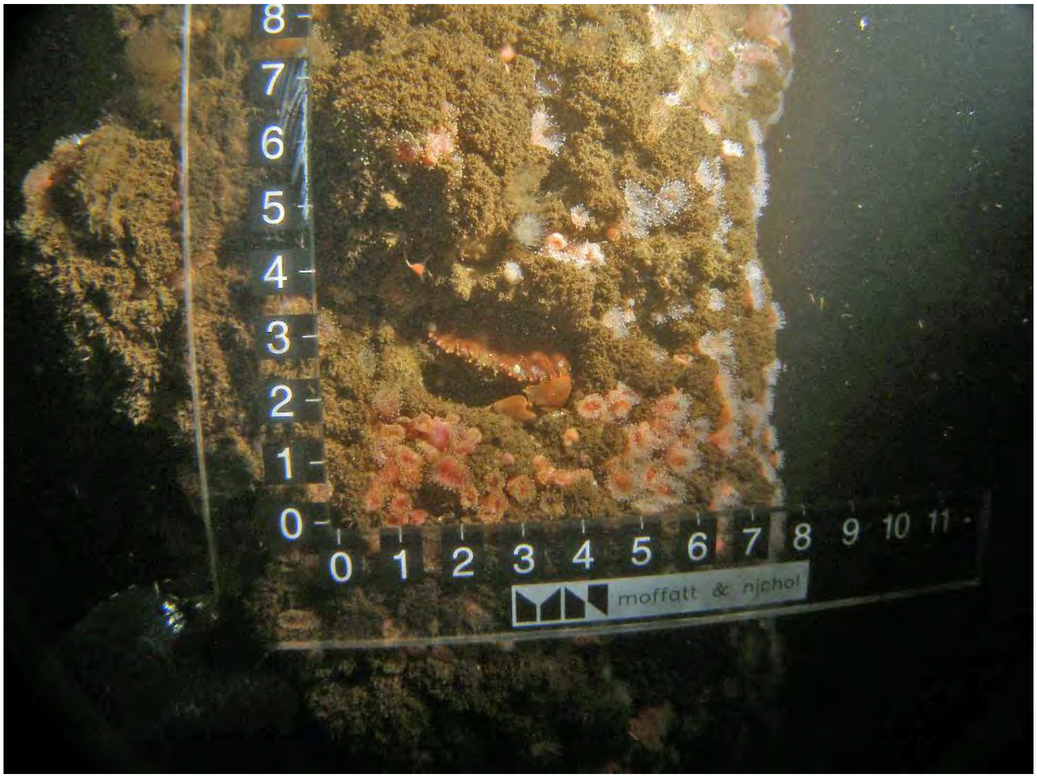

Photo 1-22: Bent 76, Pile 14 – Portion of pile with 80% section loss (Deep hollow hole with crab living inside)

Photo 1-23: Bent 112, Pile 5 - Hole with 30%-80% Section Loss

Photo 1-24: Bent 42, Pile 6 – 90% section loss

Photo 1-25: Bent 178, Pile 14 – Hole on pile with 90% section loss (not shown in picture)

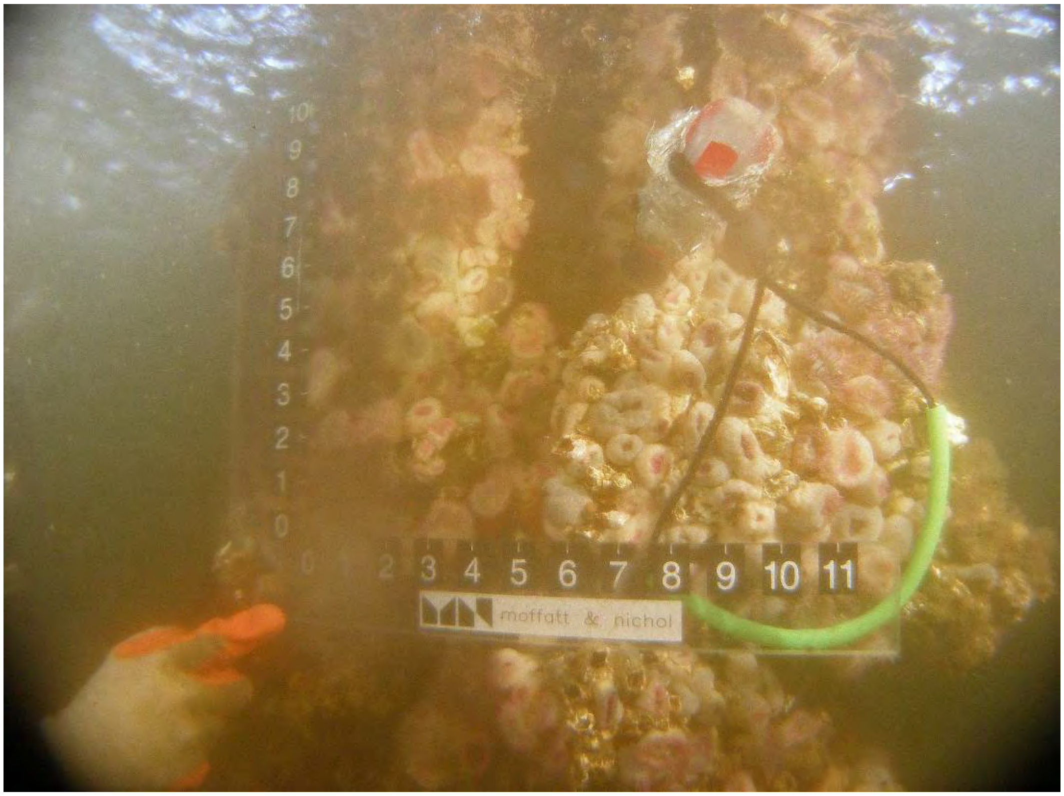

Photo 1-26: Bent 179, Pile 13 – Pile with 90% section loss

Photo 1-27: Bent 78, Pile 20 – 95% section loss

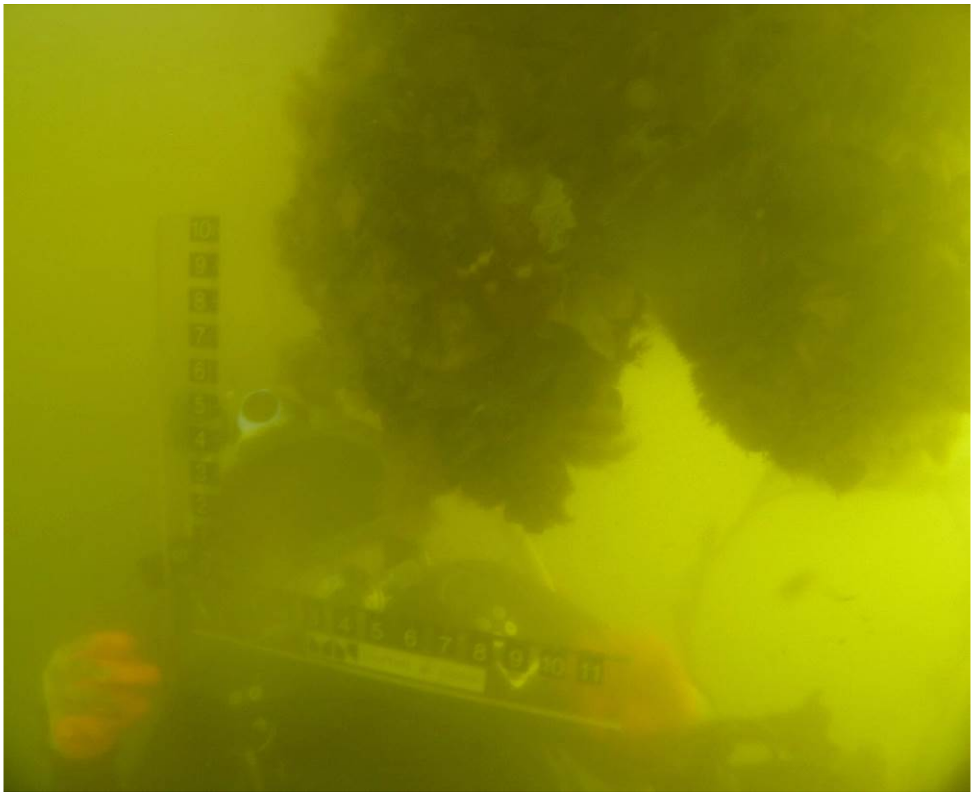

Photo 1-28: Bent 179, Pile 2 – Pile with 100% section loss

10/03/2013 17:04

Photo 1-29: Bent 42 pile stub

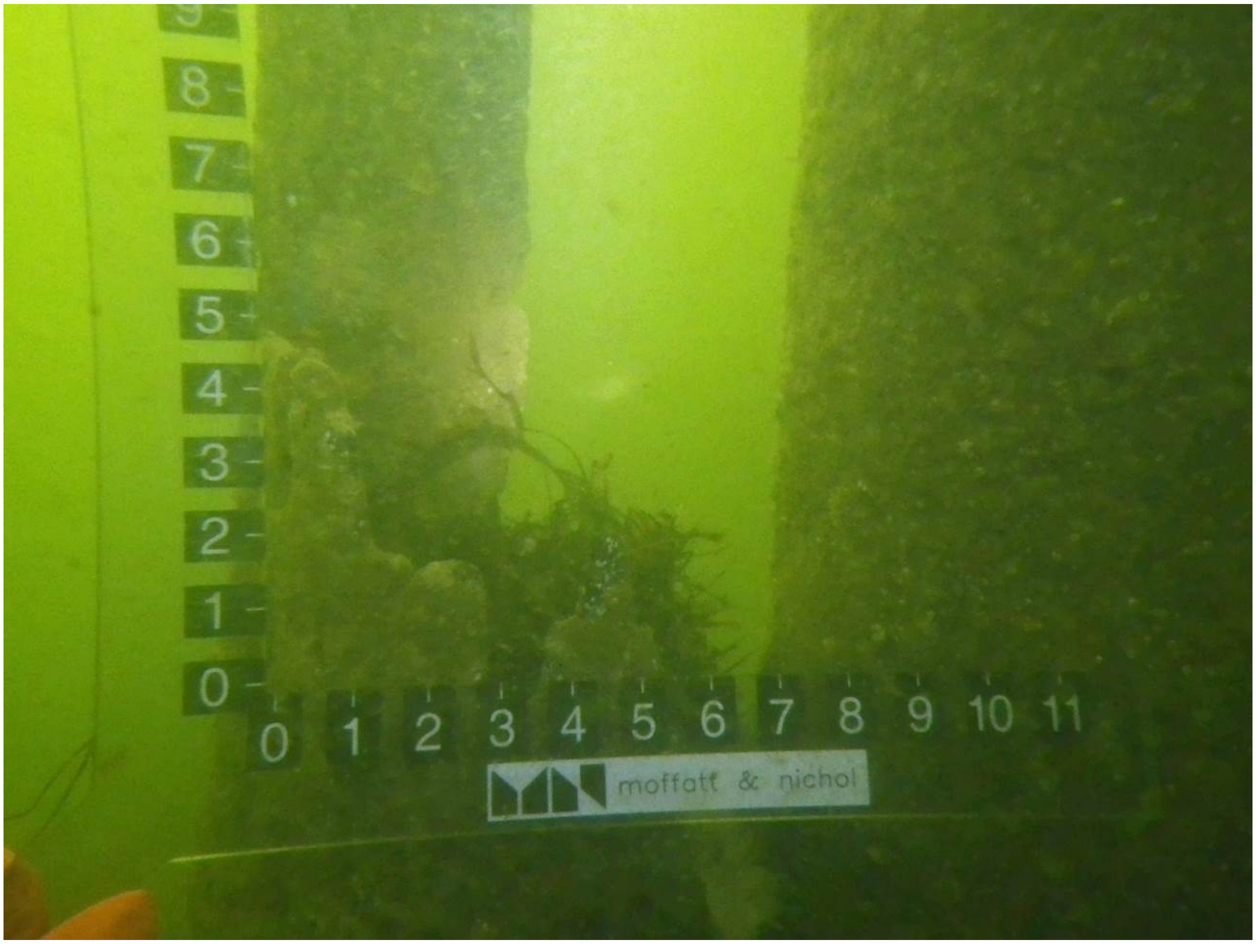

Photo 1-30: Bent 42 pile stub

Photo 1-31: Composite photo of Bent 137A, ~14 ft. span and damaged piles 12-14 (below water)

Photo 1-32: Multiple A-frames under Miramar

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Photo 1-34: Ideal splice cap and T- connection (Bent 122, Pile 2)

Photo 1-35: U-strap connection (Bent 152)

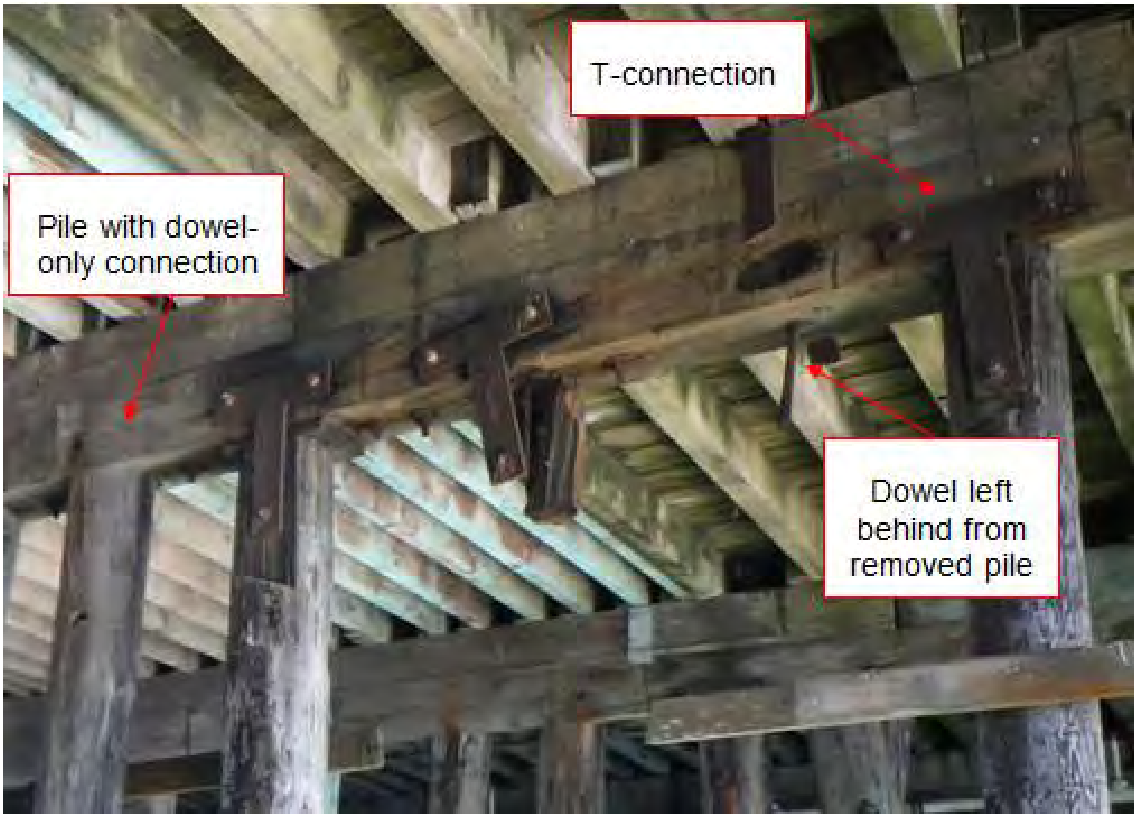

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Photo 1-60: Bent 117 Pile 1

Photo 1-61: Bent 117 Pile 4

Photo 1-62: Bent 117 Pile 24

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Photo 1-64: Bent 118 Pile 18

Photo 1-65: Bent 118 Pile 33

Photo 1-66: Bent 126 Pile 5

Photo 1-67: Bent 126 Pile 15

Photo 1-68: Bent 126 Pile 34

Photo 1-69: Bent 137 East Pile 15

Photo 1-70: Bent 138 East Pile 7

Photo 1-71: Bent 138 West Pile 13

Photo 1-72: Bent 139 West Pile 10

Photo 1-73: Bent 155 Pile 33

Photo 1-74: Bent 163 Pile 9

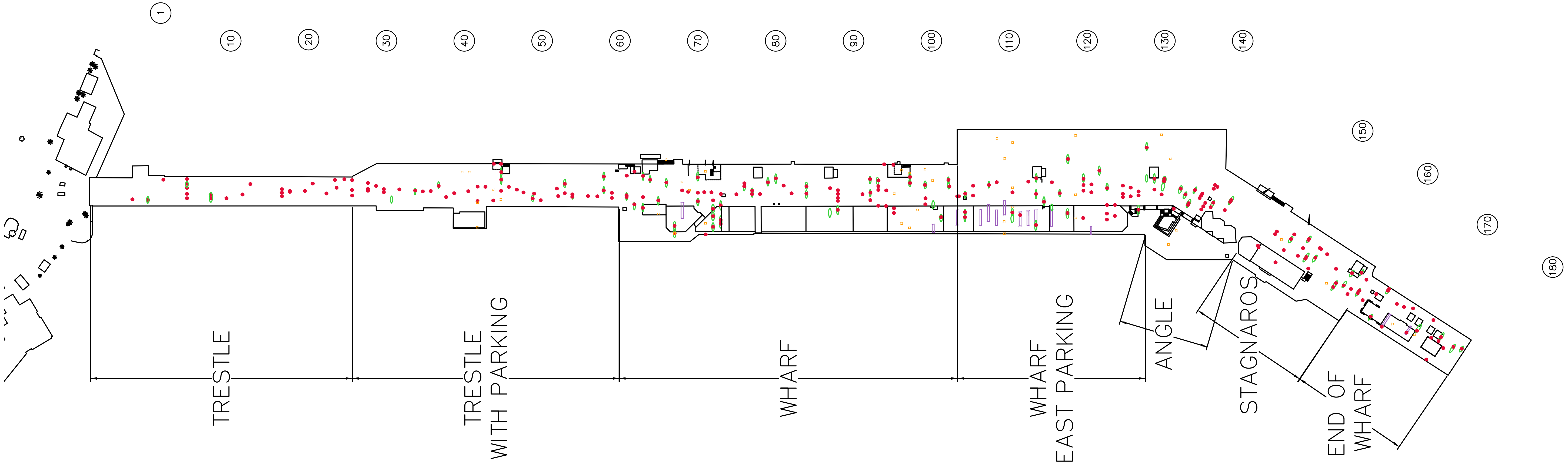
Photo 1-75: Bent 173 Pile 5

Photo 1-76: Bent 176 Pile 9

Attachment 1B Dive Inspection Data Tables

(Provided on CD)

Attachment 1C Damaged Pile Plans



KEY

● = DAMAGED PILE WITH >40% CROSS SECTIONAL AREA LOSS

▭ = A–FRAME

⬭ = EFFECTIVE PILE SPAN >10 FT.

□ = PILE CORE LOCATION

SANTA CRUZ WHARF PLAN

SCALE: 1"= 100'

100' 0' 100' 200'

SCALE: 1"=100'

Layout Tab Name: Damaged Piles Plan, Images: 115808.JPG: 115810.JPG: , Xrefs: Aerial.dwg: ACAD–DeckPoints.dwg
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SANTA CRUZ WHARF
SURVEY
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REFERENCES
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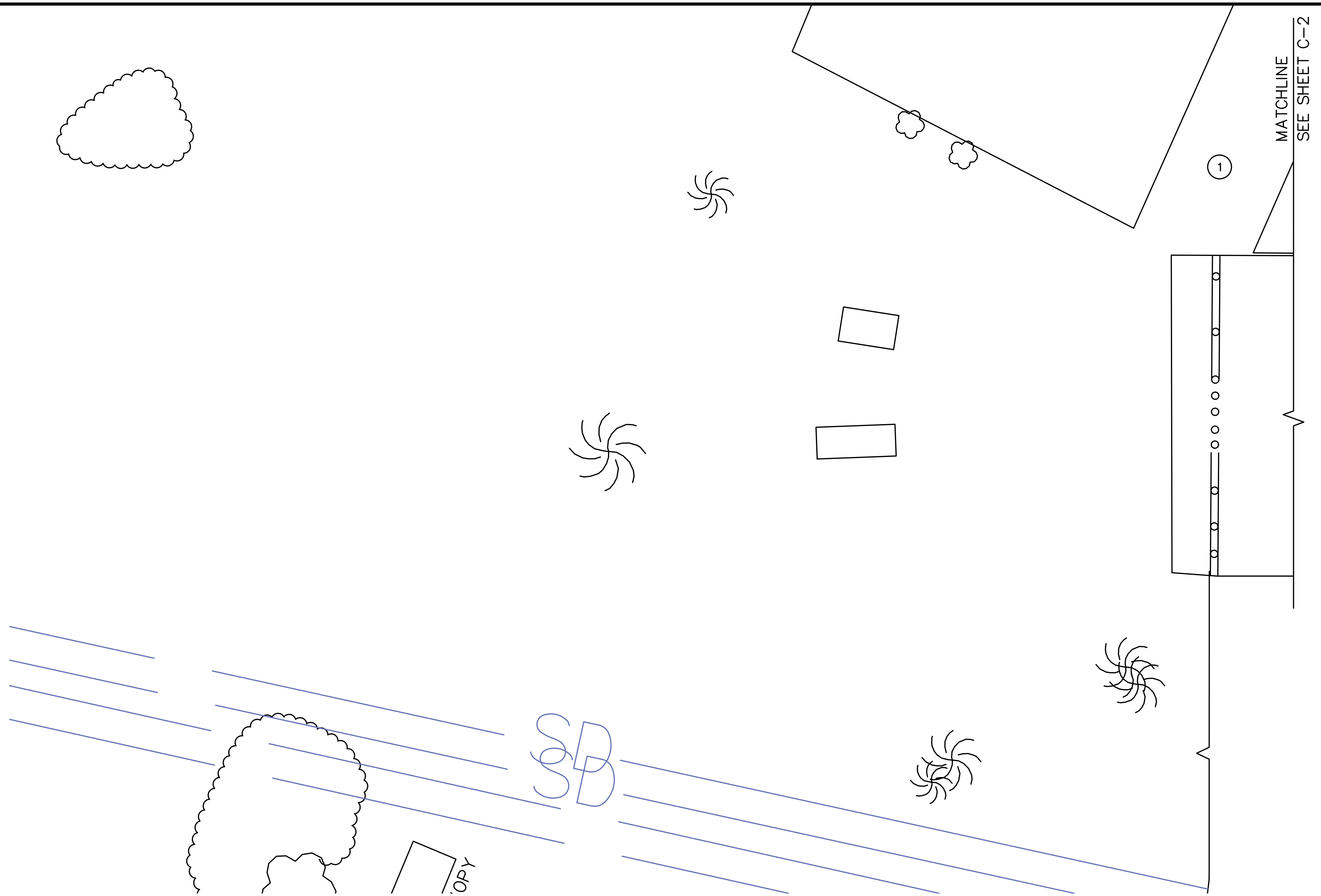

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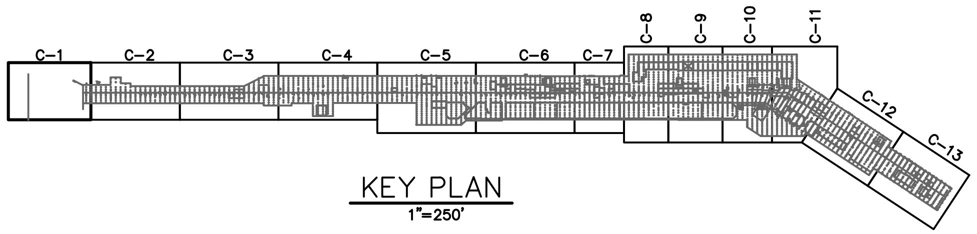

KEY PLAN
1"=250'

10' 0' 10' 20'
SCALE: 1"=10'

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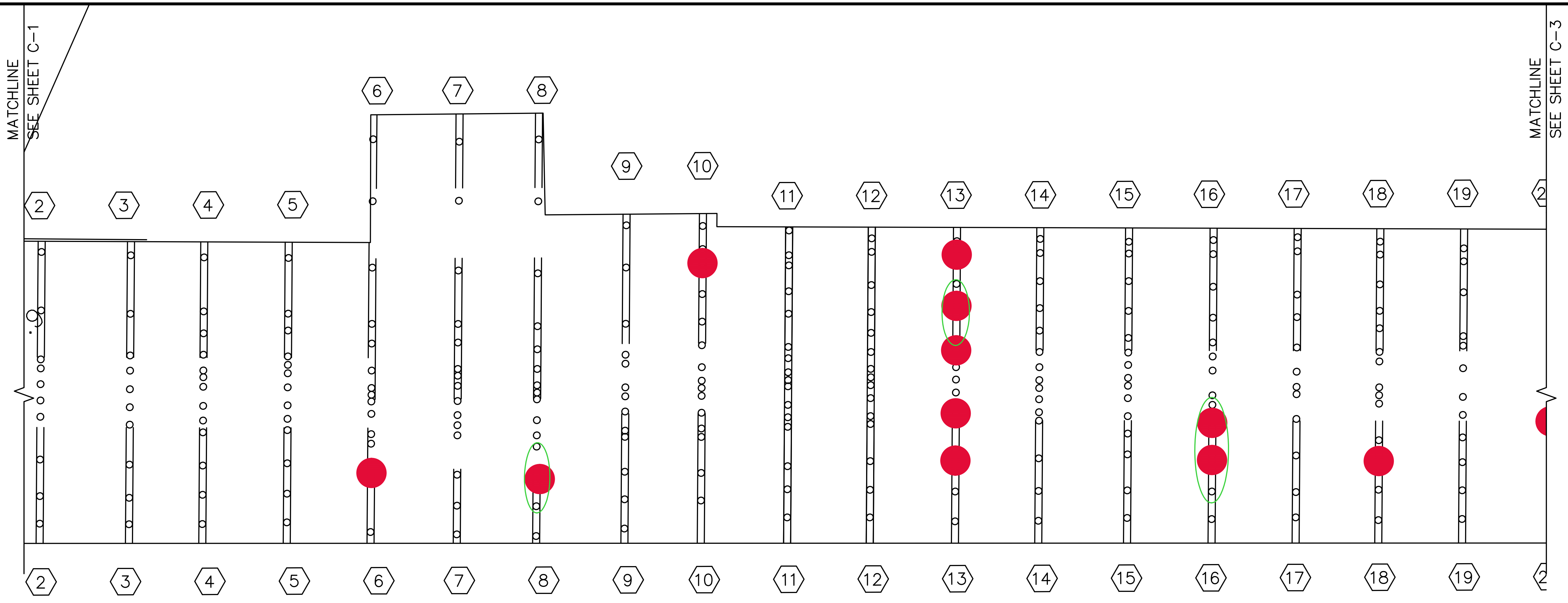

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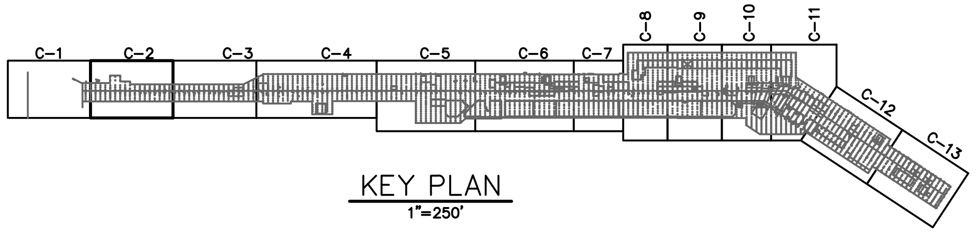

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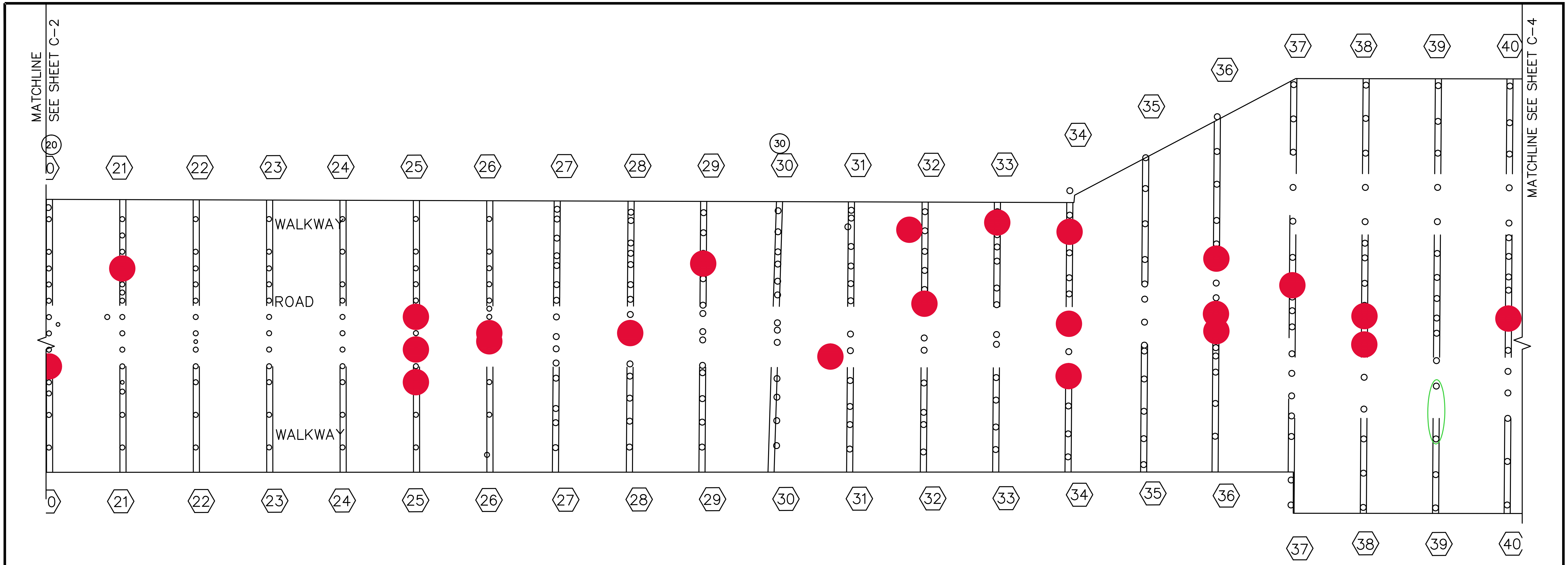

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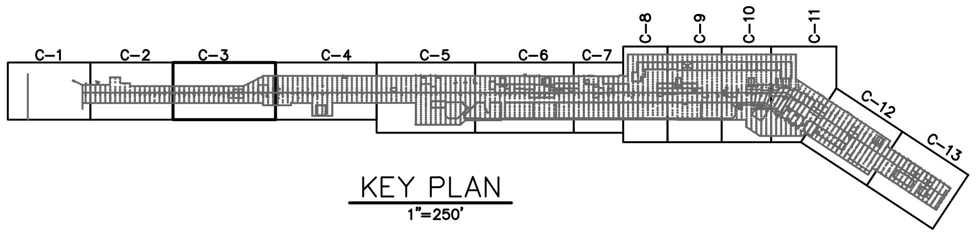

KEY PLAN

1"=250'

10' 0' 10' 20'

SCALE: 1"=10'

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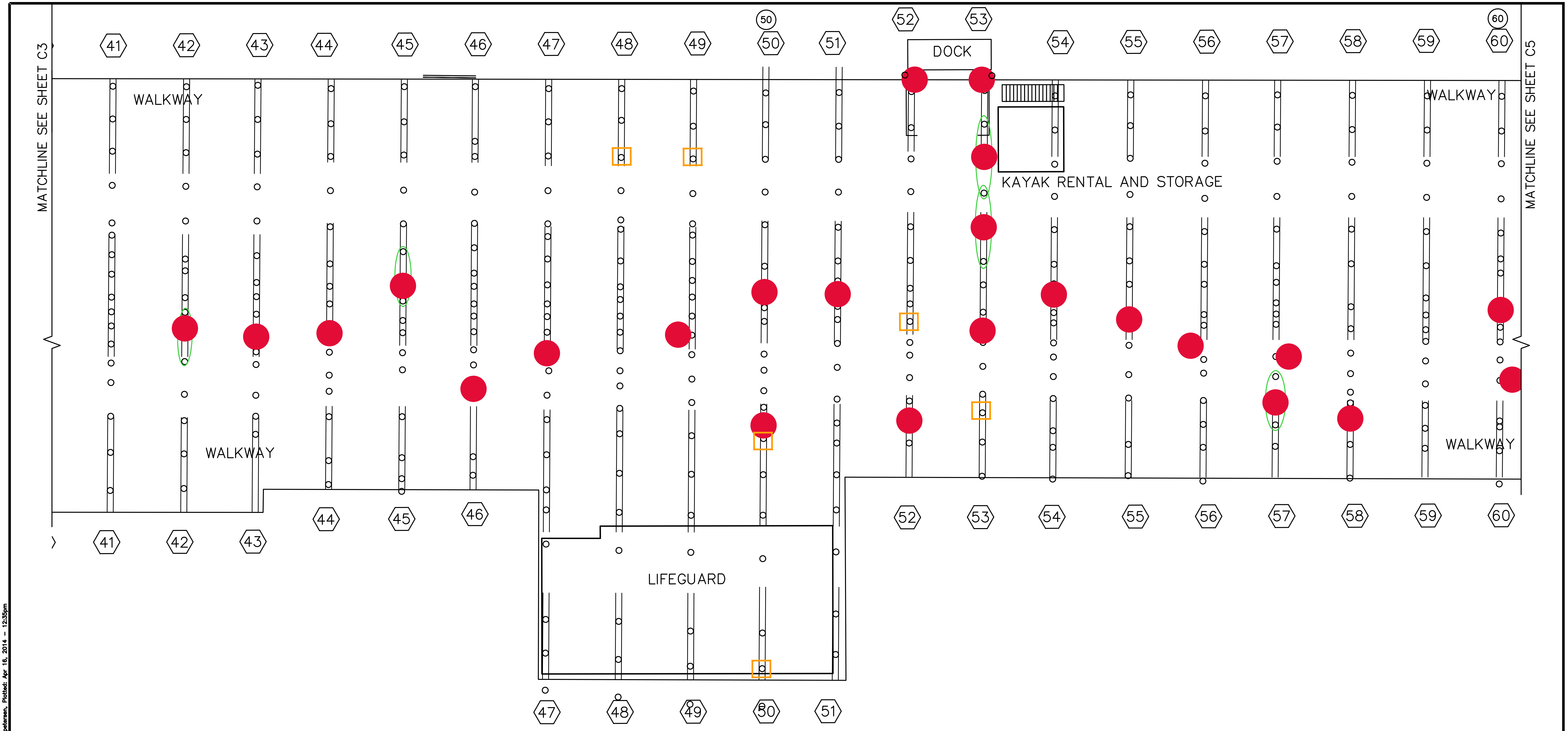

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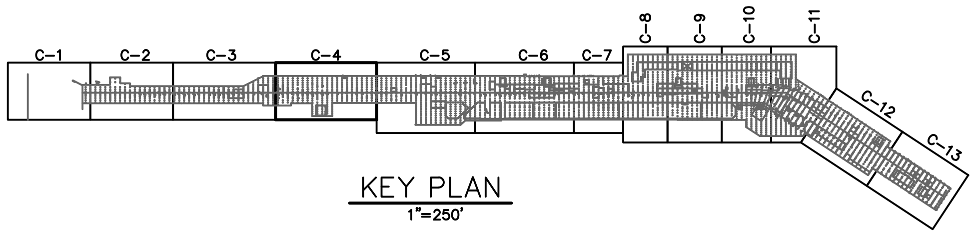

KEY PLAN

1"=250'

SCALE: 1"=10'

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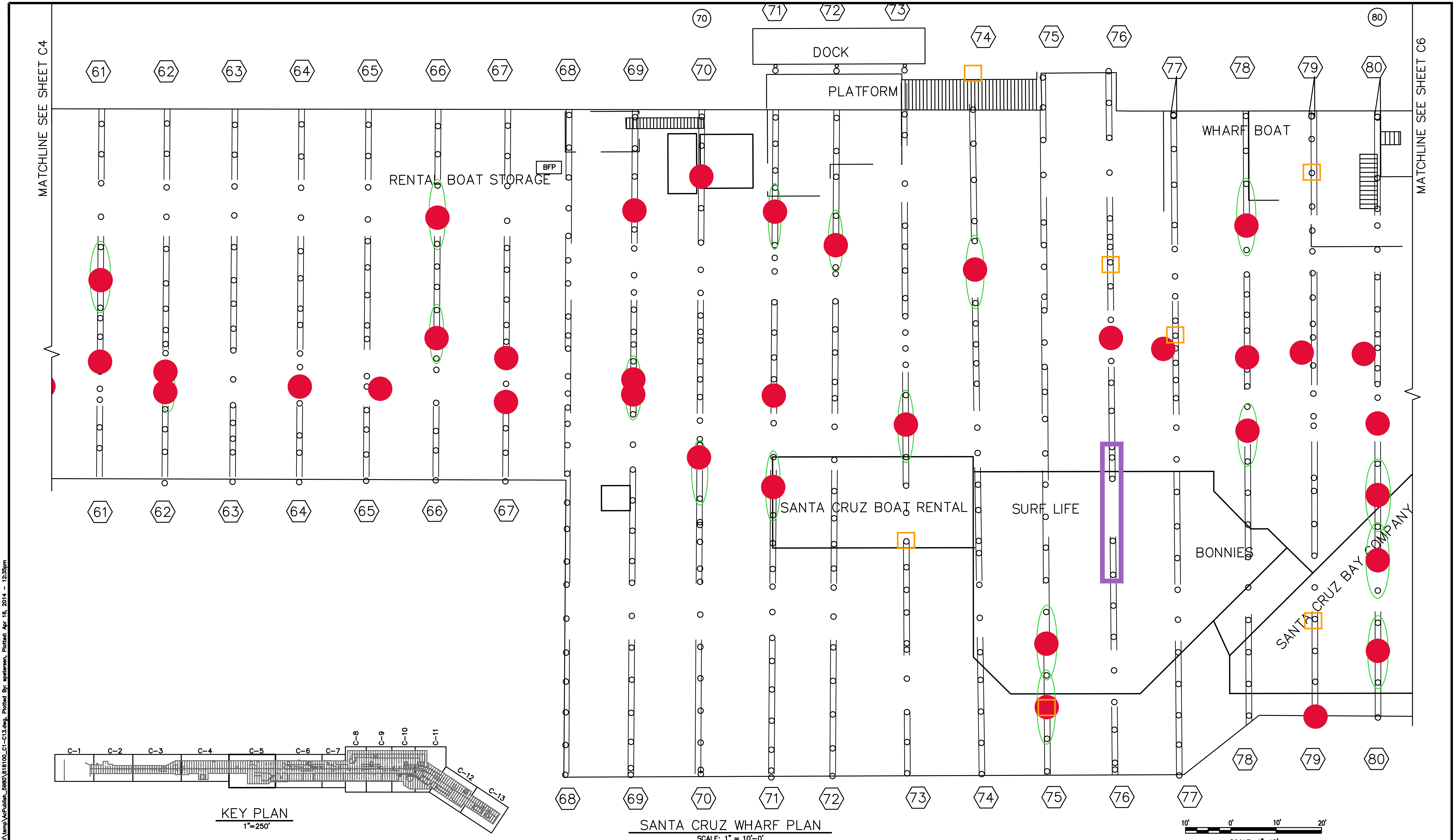

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SANTA CRUZ WHARF PLAN
SCALE: 1" = 10'-0'

SCALE: 1"=10'

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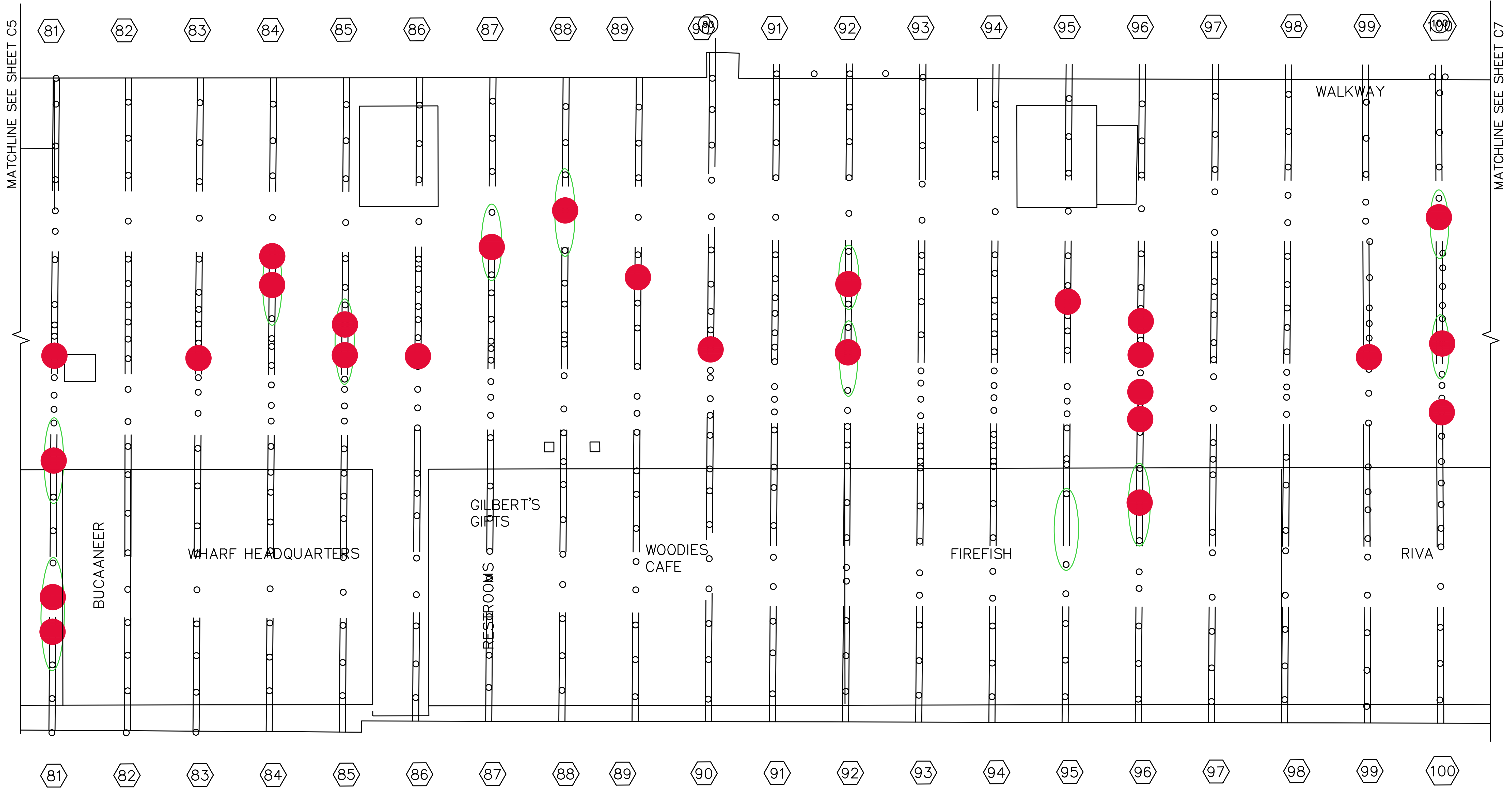

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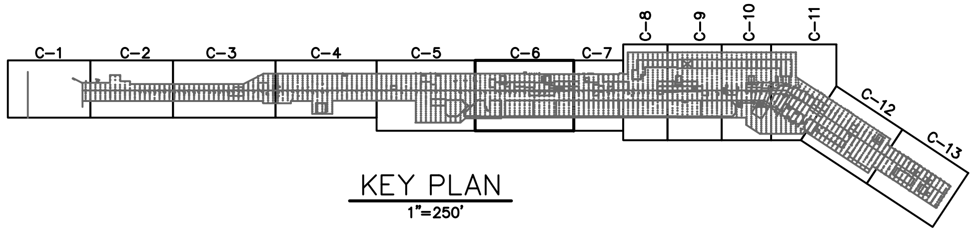

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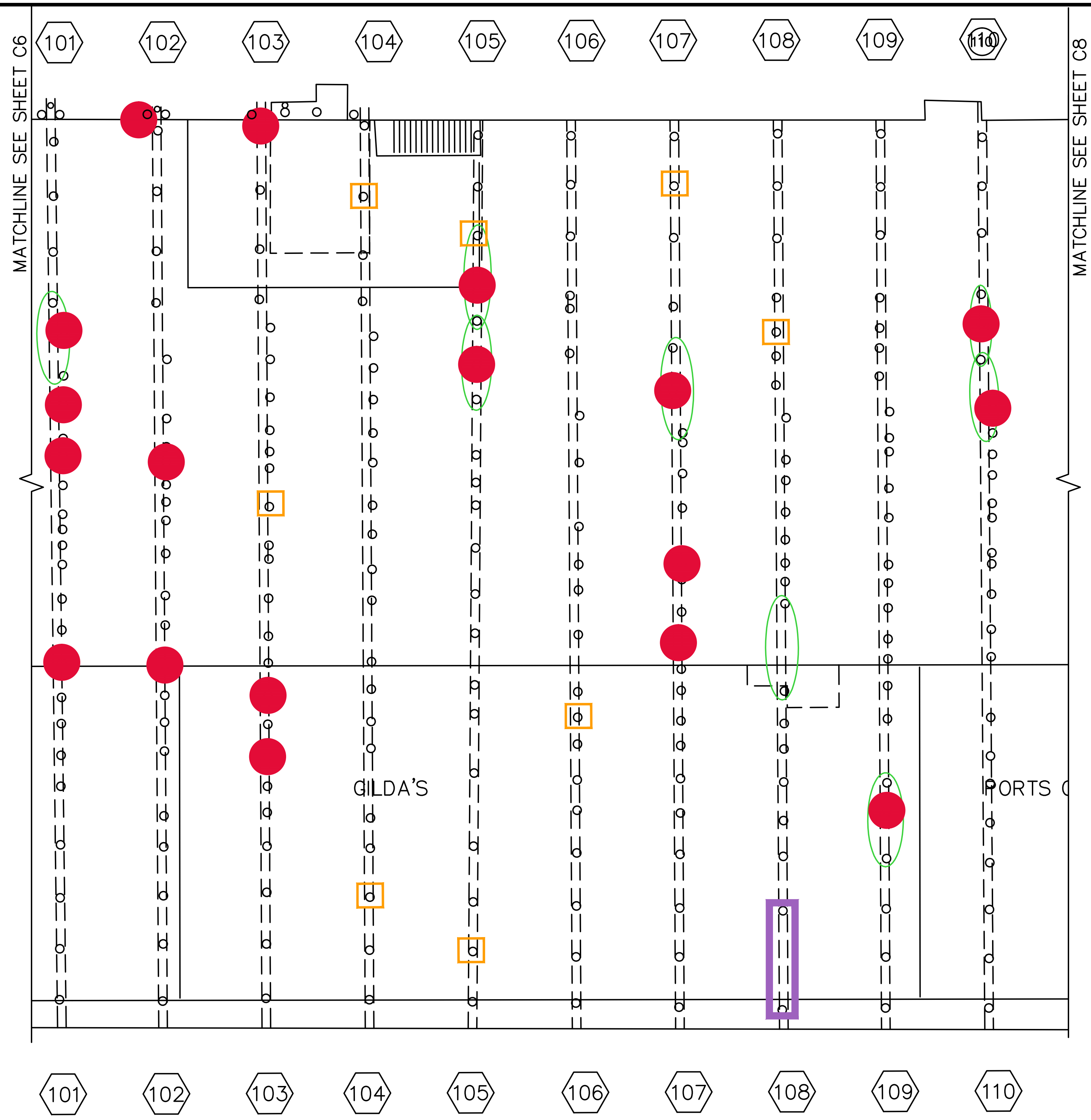

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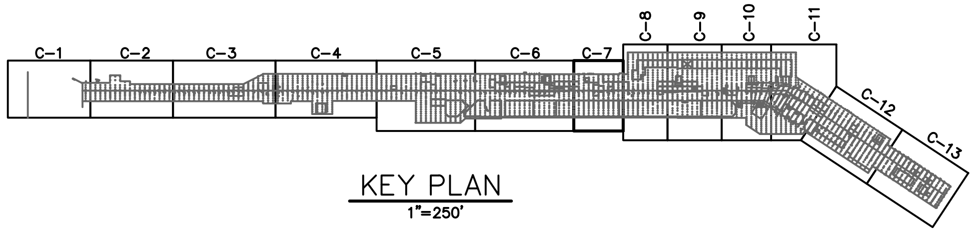

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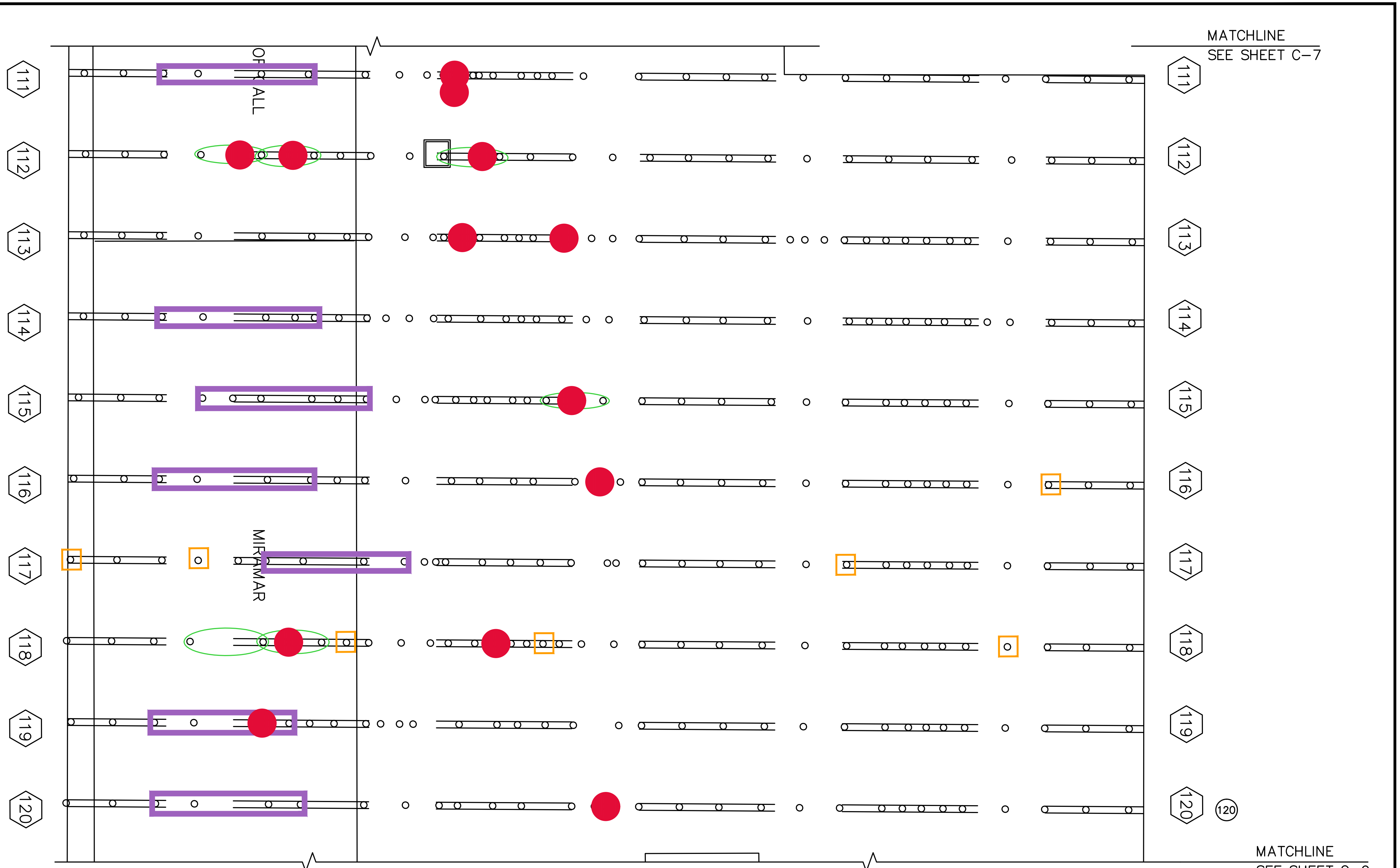

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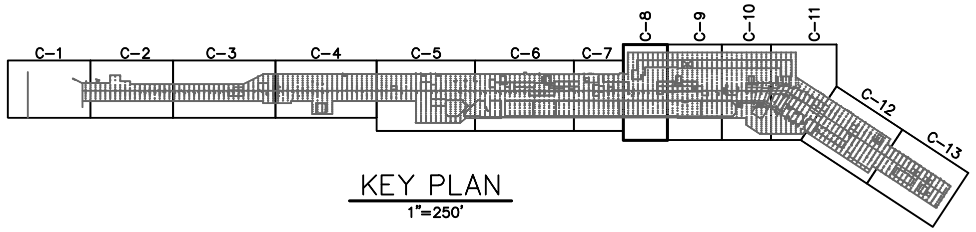

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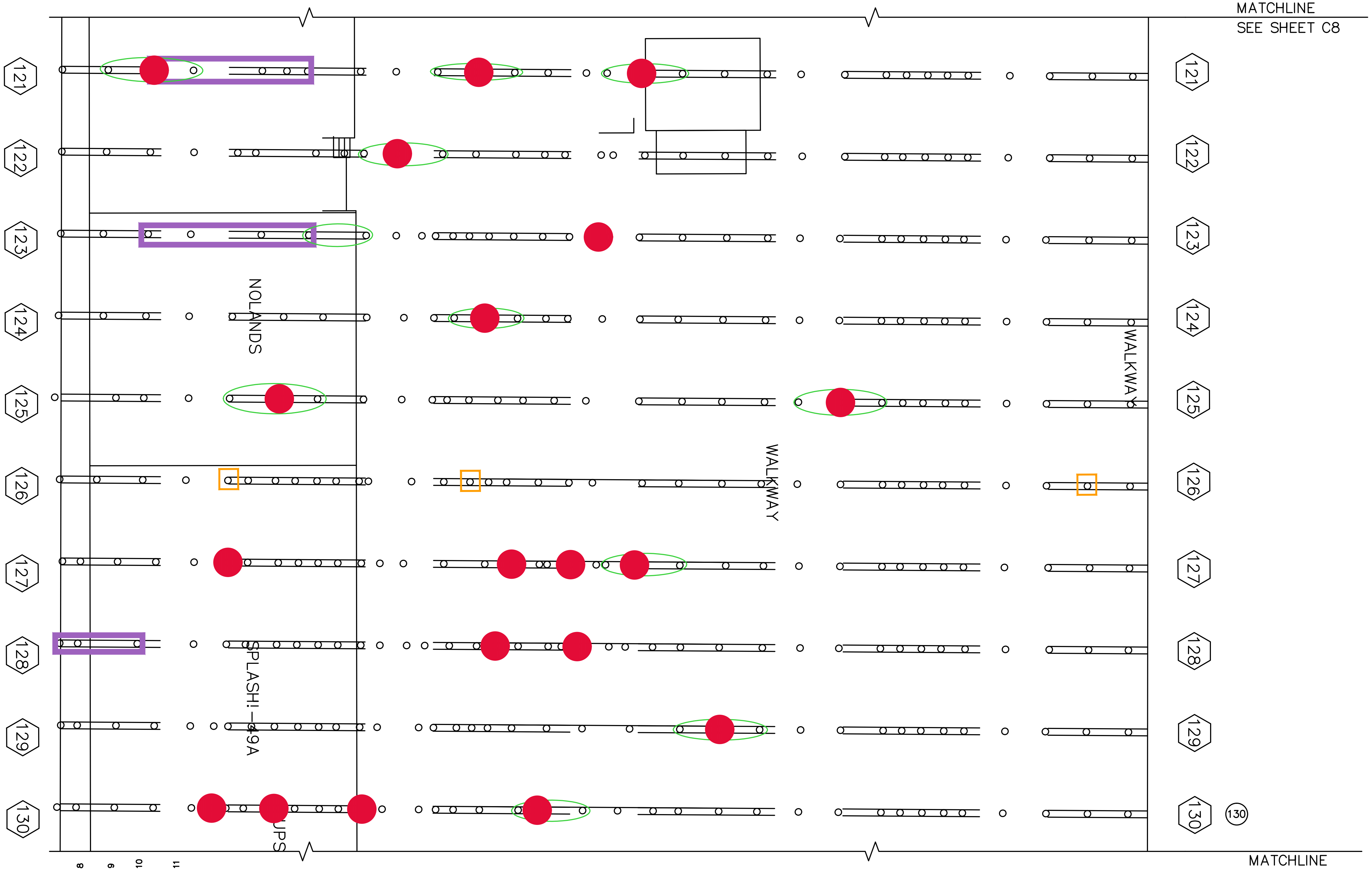

SANTA CRUZ WHARF PLAN

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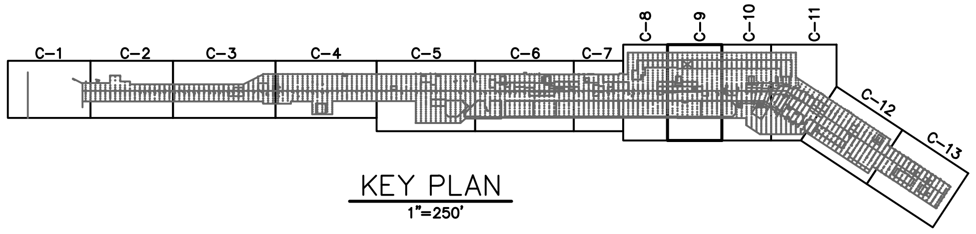

KEY PLAN

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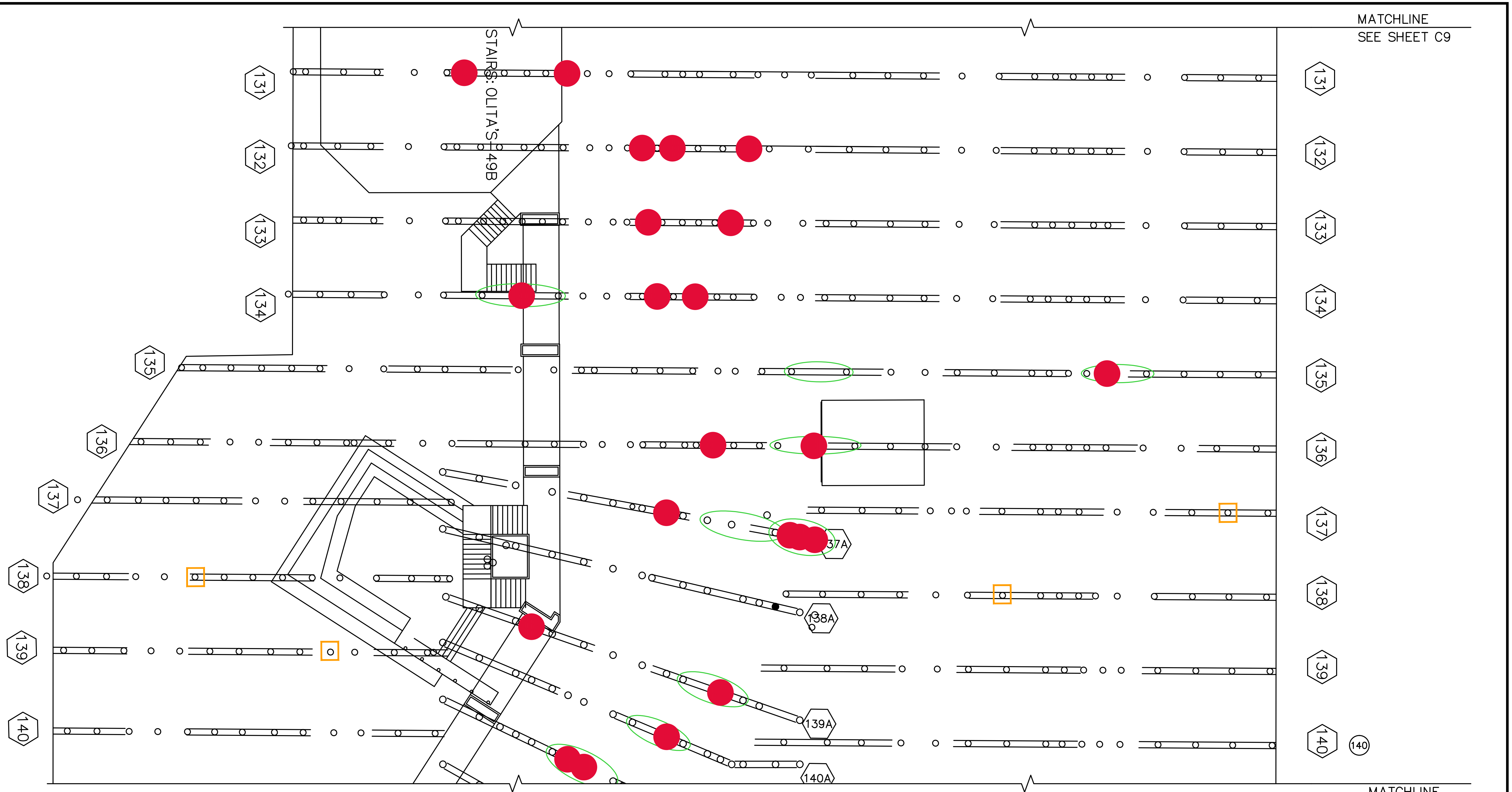

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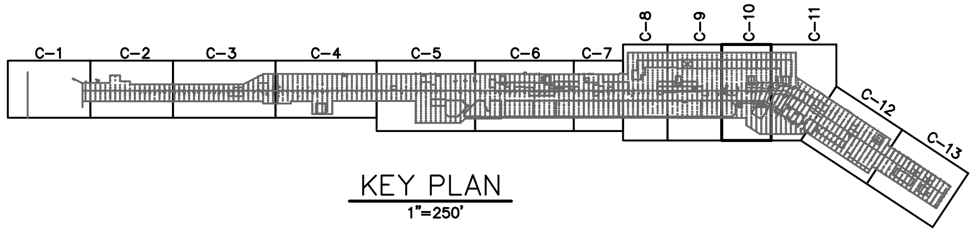

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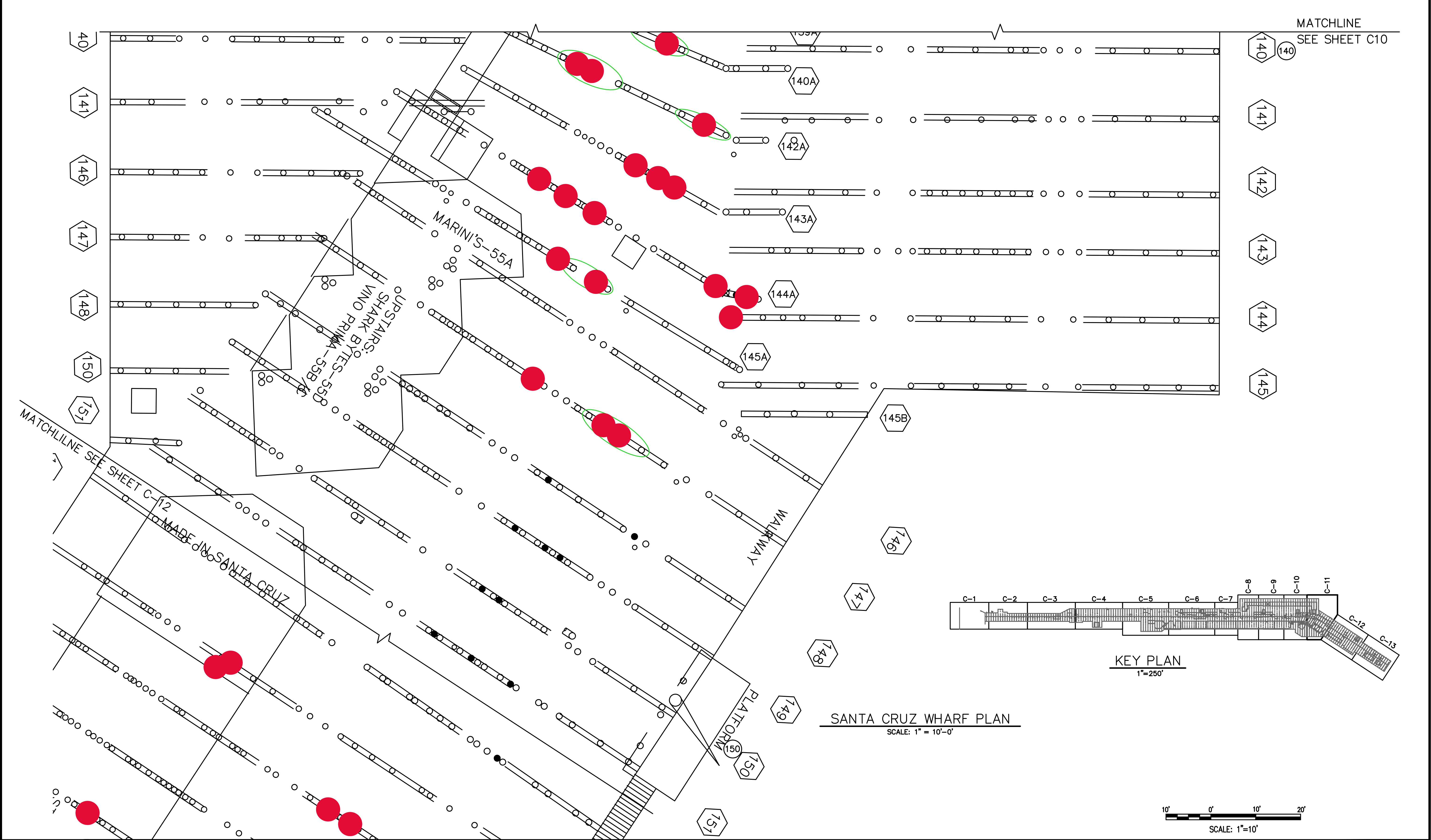

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809 Center Street, Room 201
Santa Cruz, CA 95060

SANTA CRUZ WHARF
SURVEY
DECK PLAN

REFERENCES
FIELD BOOK:
#
DRAWING #:
#

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DESIGN	BP	VAULT NO.	
CHECKED	BP	#	

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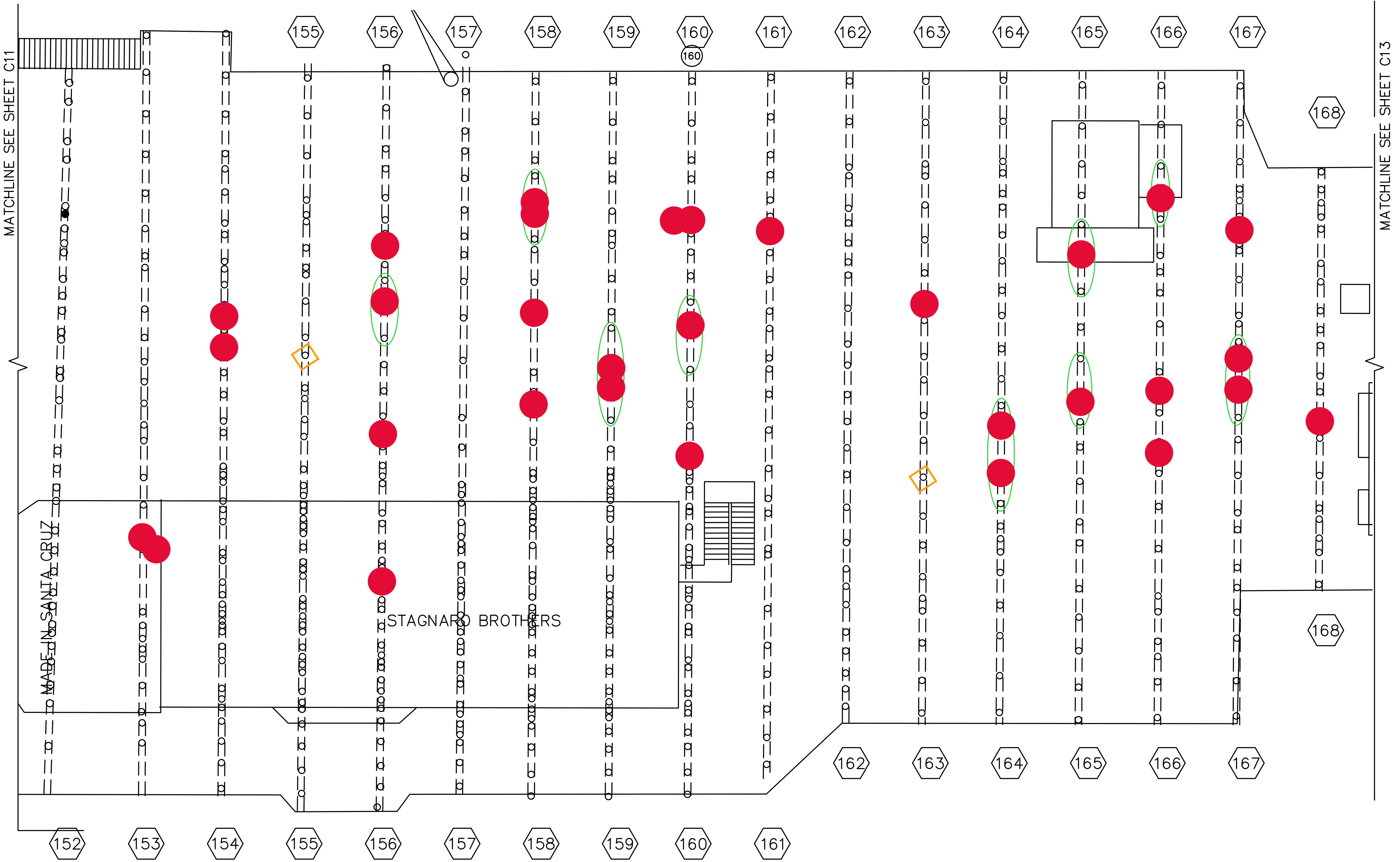
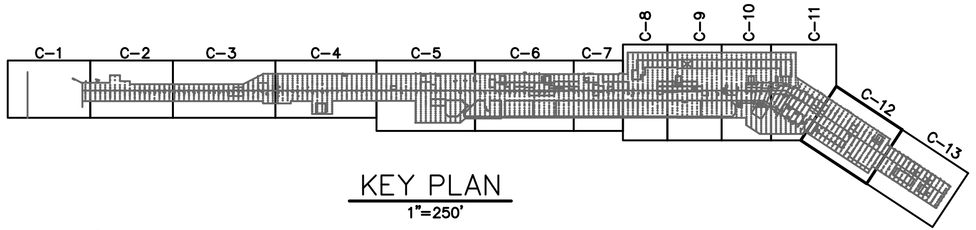


SANTA CRUZ WHARF PLAN

SCALE: 1" = 10'-0'



KEY PLAN

1"=250'

10' 0' 10' 20'

SCALE: 1"=10'

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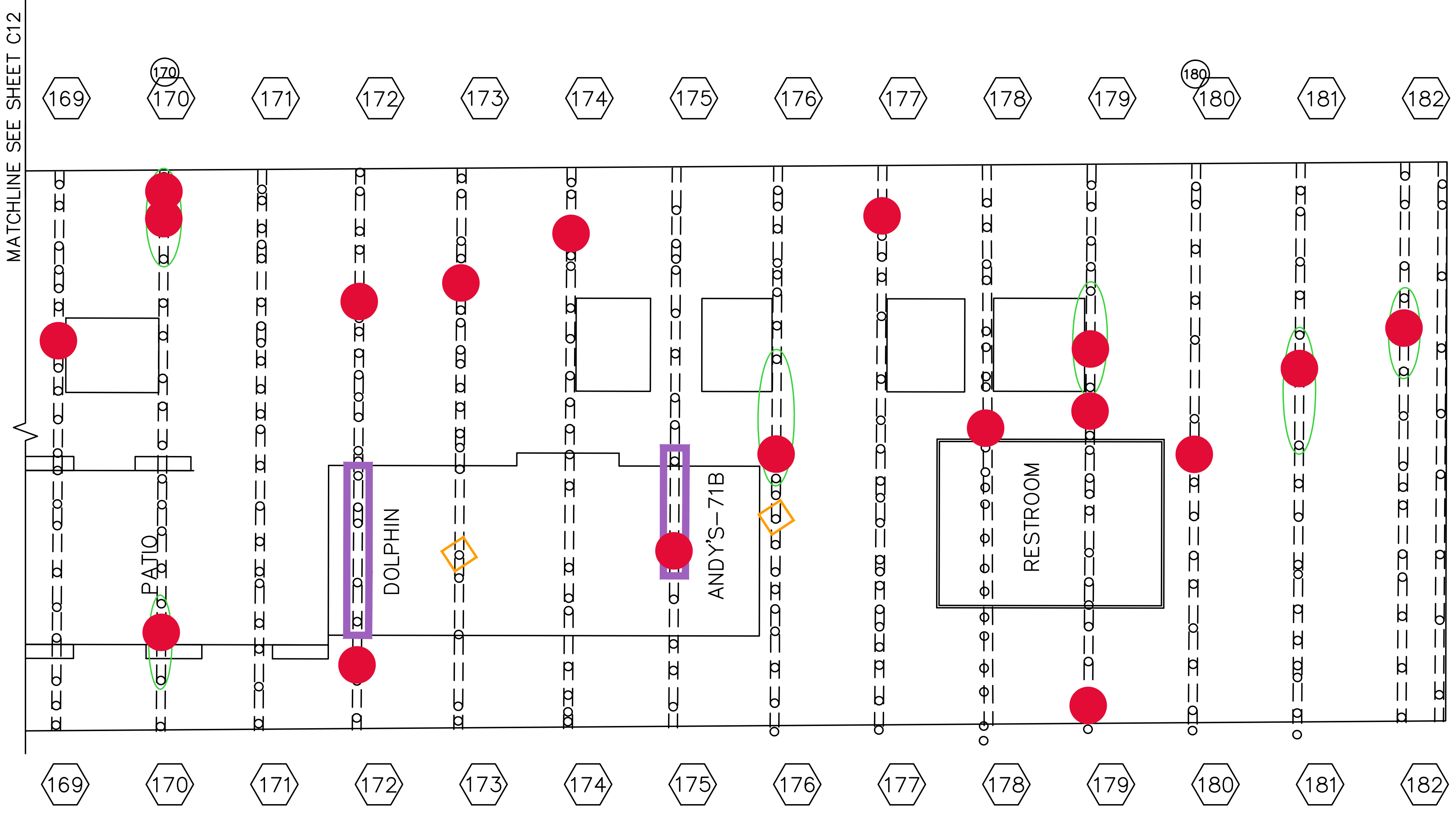
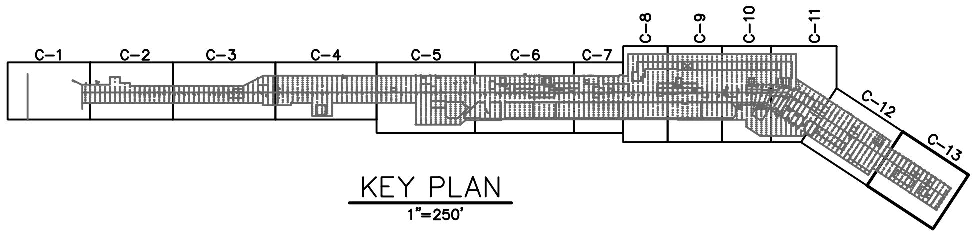
DRAWN	TE	SHEET	C12 OF -
DESIGN	BP	VAULT NO.	
CHECKED	BP	#	



SANTA CRUZ WHARF PLAN

SCALE: 1" = 10'–0'



KEY PLAN

1"=250'

10' 0' 10' 20'

SCALE: 1"=10'

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DATE	6-15-11	SCALE	1"=10'-0"
DRAWN	TE	SHEET	C13 OF -
DESIGN	BP	VAULT NO.	#
CHECKED	BP		

Attachment 2A Photographs

Photo 2-15: Rot on east stringers, Bent 54-55

Photo 2-16: Leaking Pipe, between Bent 103-104 west side

Photo 2-17: Approximate 9 ft. span and fire damage to cap and stringers, Bent 104

Photo 2-18: Rot between Bent 108-109, west side

Photo 2-19: Rot from pipe leak Bents 112-113 (west side)

Photo 2-20: Rot between Bent 118-119 (west side)

Photo 2-21: Rotten, crushed stringer between Bent 120-121

Photo 2-22: Cracks in Stringers on East side of Bents 150-151

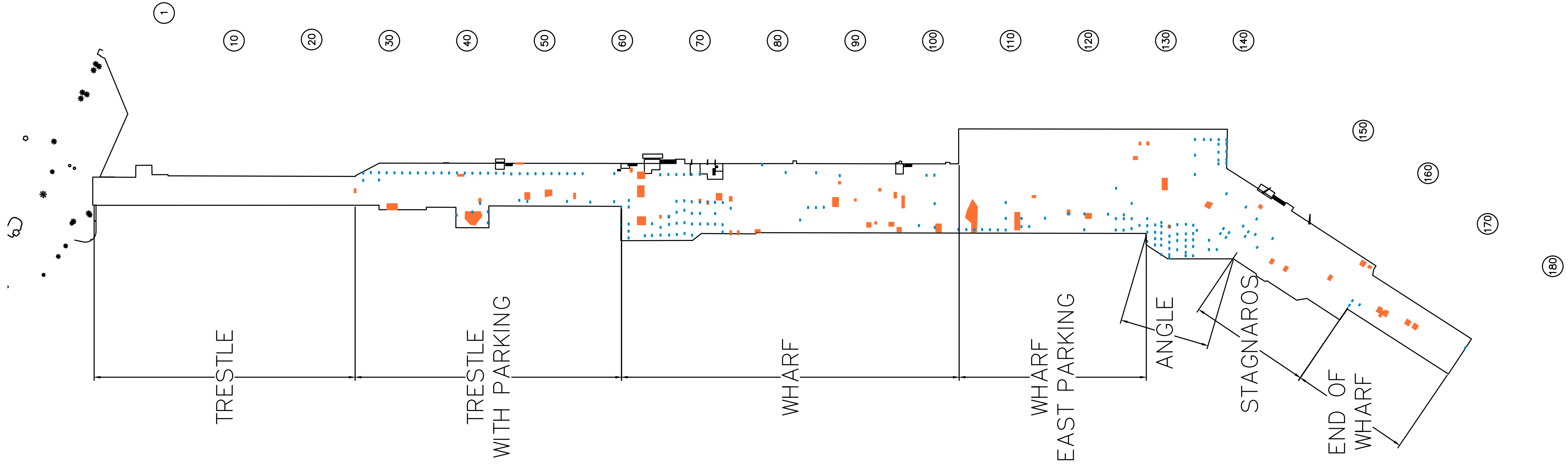
Photo 2-23: Rot between 165-166 (east side)

Photo 2-24: Bent 108 - Approx. 13 ft. span & unsupported cap splice.

Photo 2-25: Unsupported splice cap (Bent 113, Piles 1-2)

Photo 2-26: Bents 137 – 140 (west side), multiple unsupported cap splices

Attachment 2B Damaged Substructure Plans



SANTA CRUZ WHARF PLAN

SCALE: 1"= 100'

KEY

🟧 = DECK/CAP/STRINGER ROT OR DAMAGE

☐ = UNSUPPORTED CAP SPLICE

100' 0' 100' 200'

SCALE: 1"=100'

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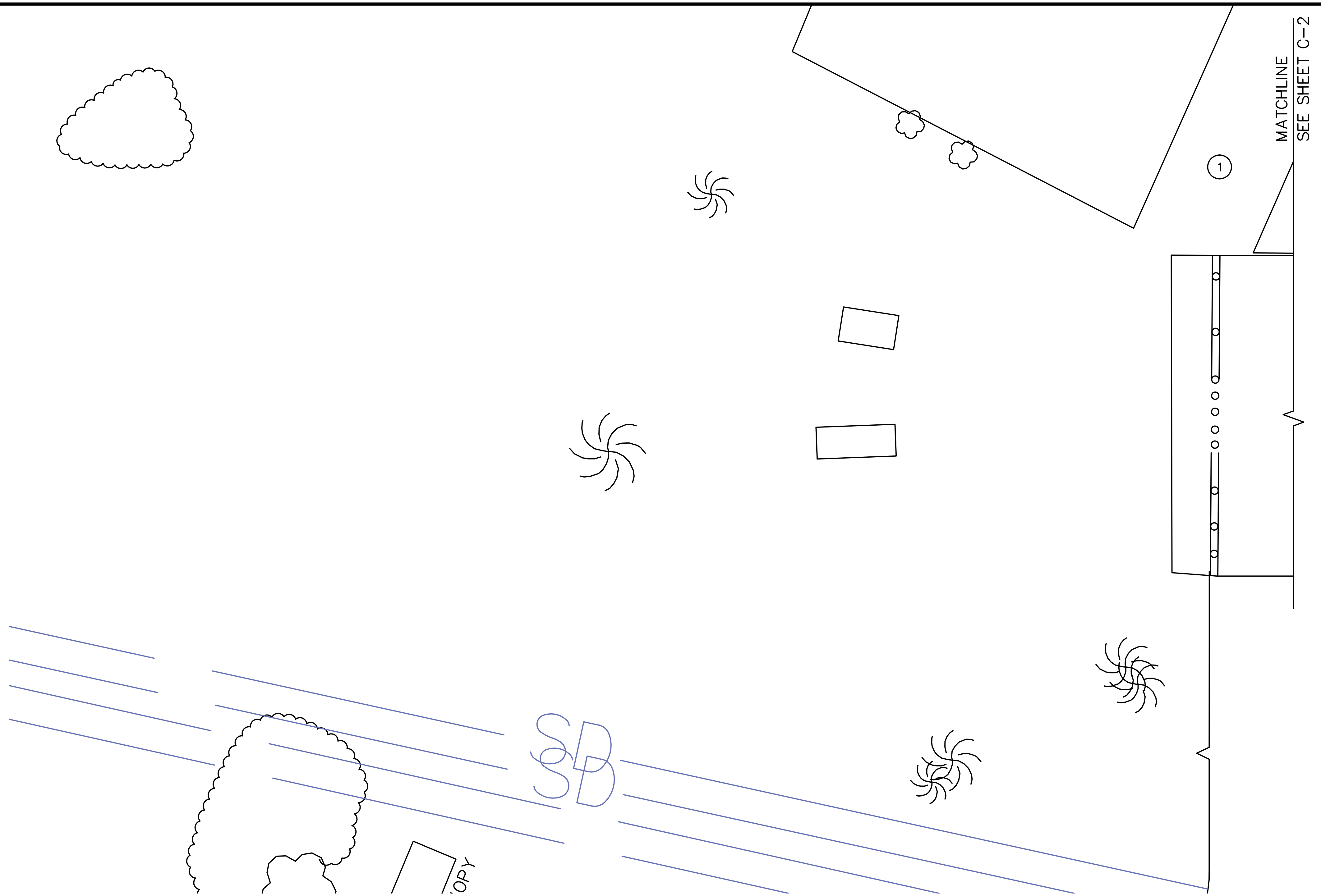

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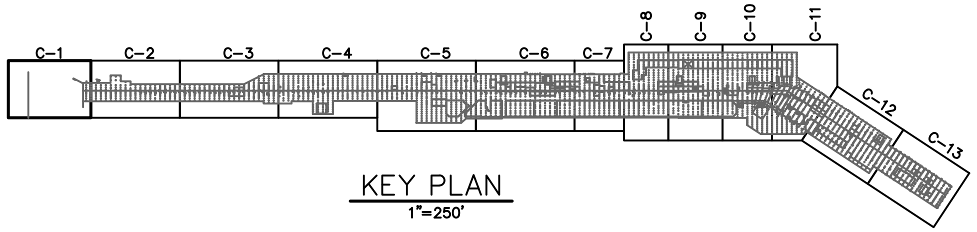
SCALE: 1" = 10'-0'



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10' 0' 10' 20'

SCALE: 1"=10'

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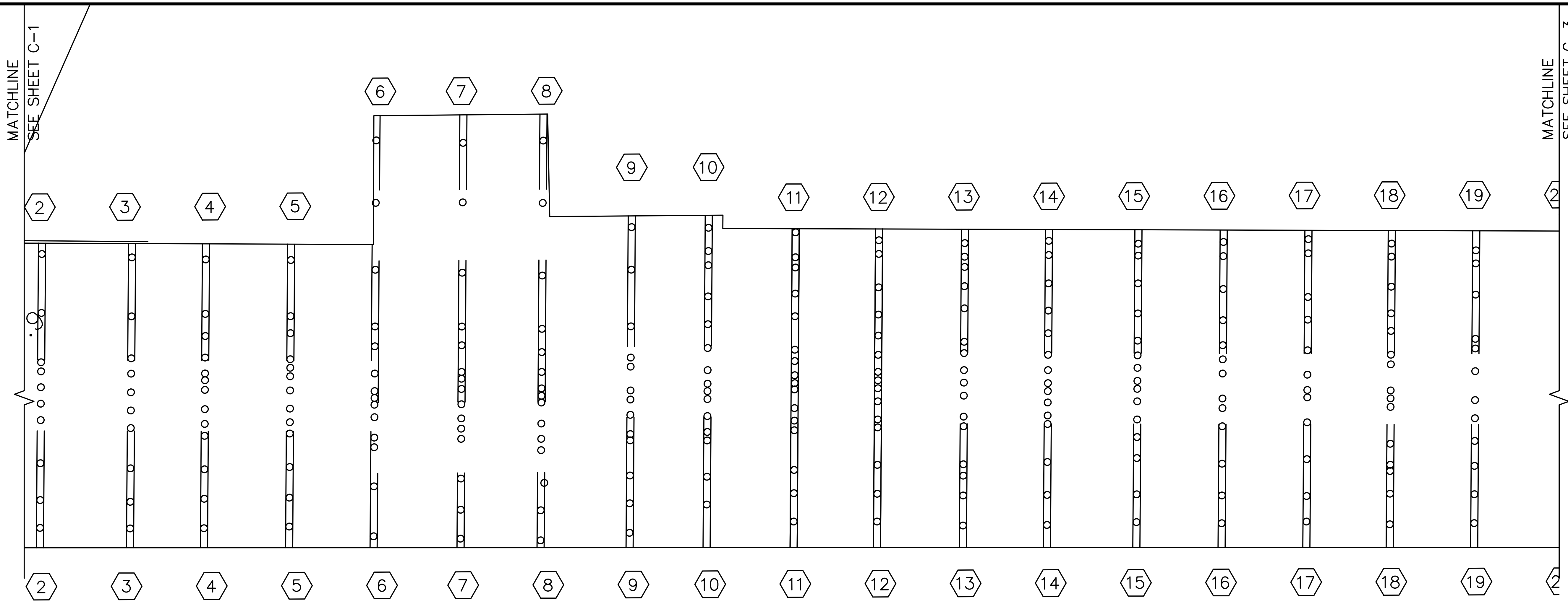
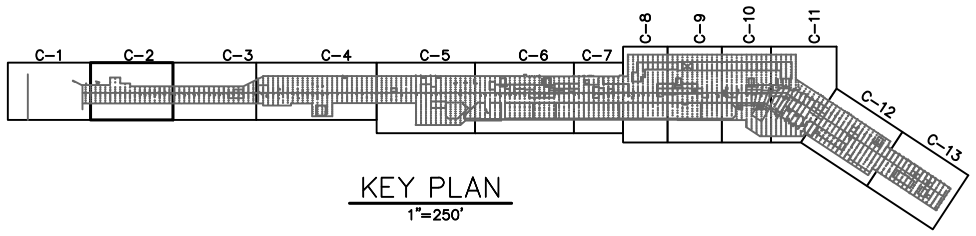


SANTA CRUZ WHARF PLAN

SCALE: 1" = 10'-0"



KEY PLAN

1"=250'

SCALE: 1"=10'

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FIELD BOOK: #
DRAWING #: #

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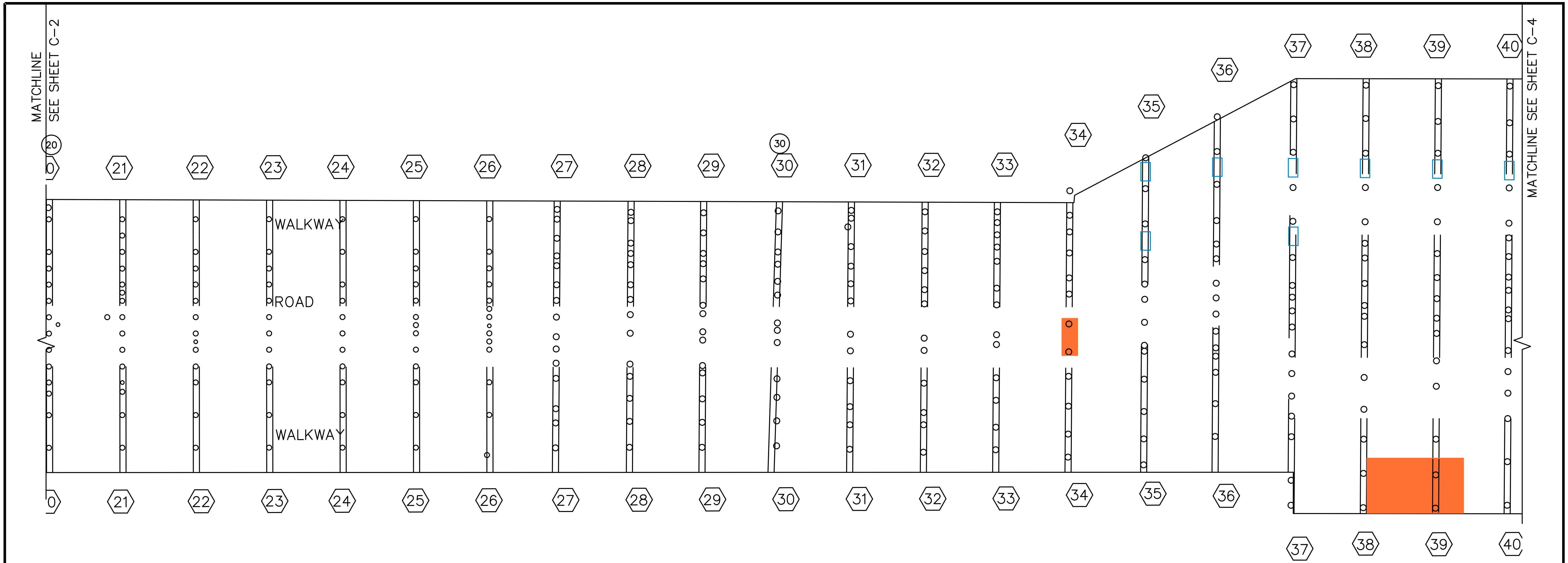
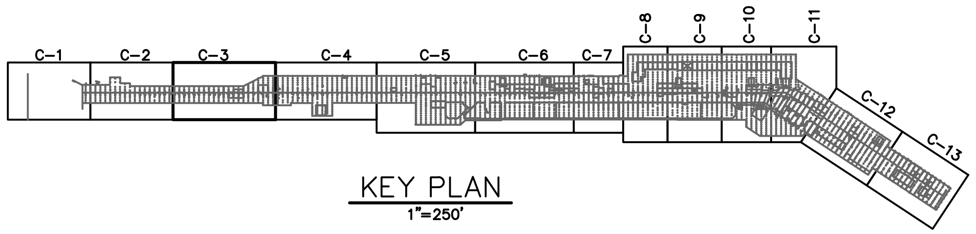
DESIGN	BP	VAULT NO.	#
CHECKED	BP		



SANTA CRUZ WHARF PLAN

SCALE: 1" = 10'-0'



KEY PLAN

1"=250'

SCALE: 1"=10'

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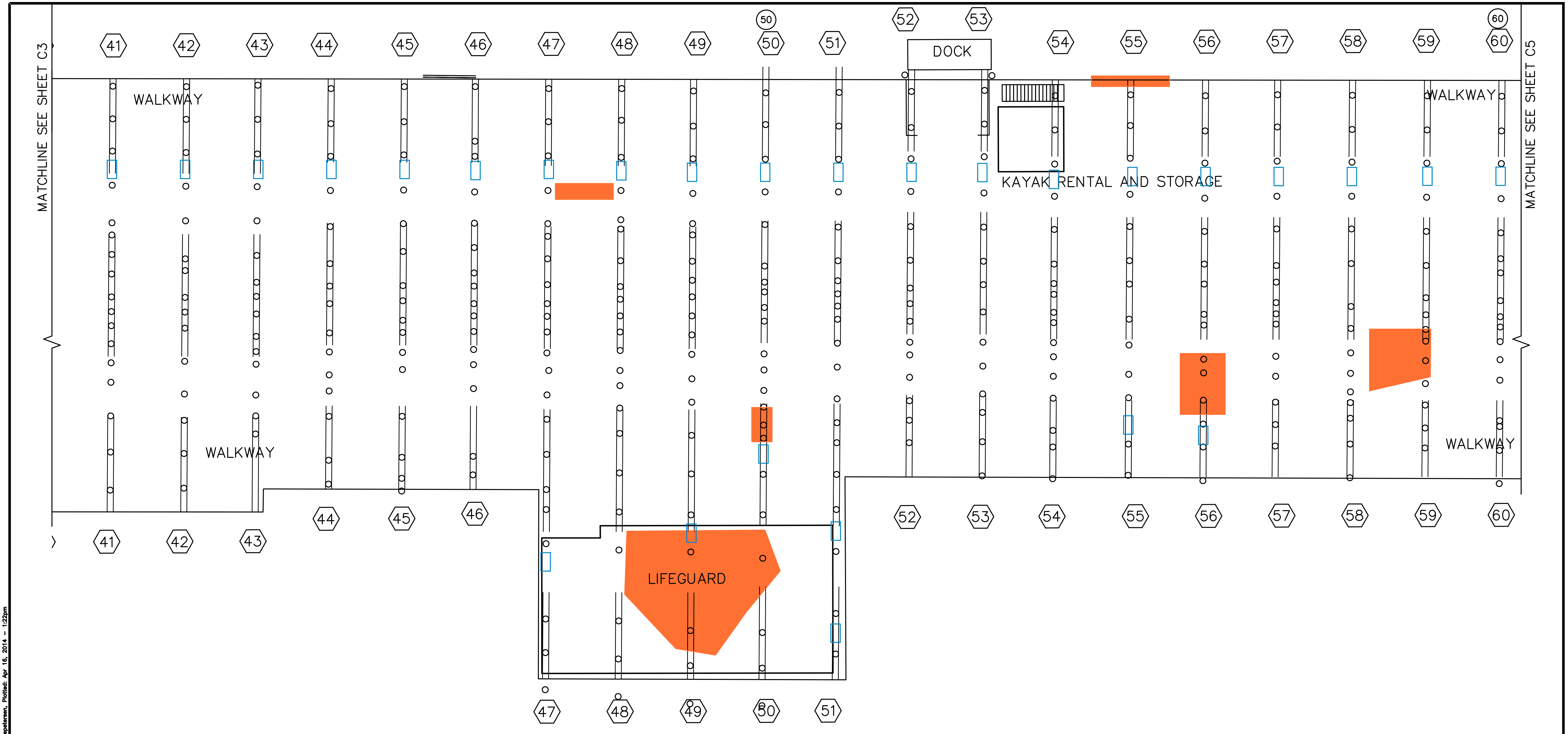
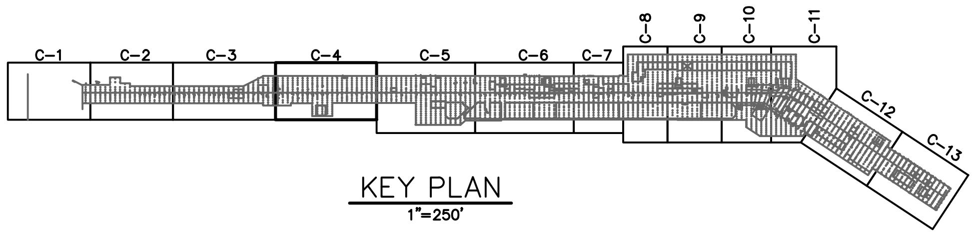


SANTA CRUZ WHARF PLAN

SCALE: 1" = 10'-0"



KEY PLAN

1"=250'

10' 0' 10' 20'

SCALE: 1"=10'

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FIELD BOOK:
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DRAWING #:
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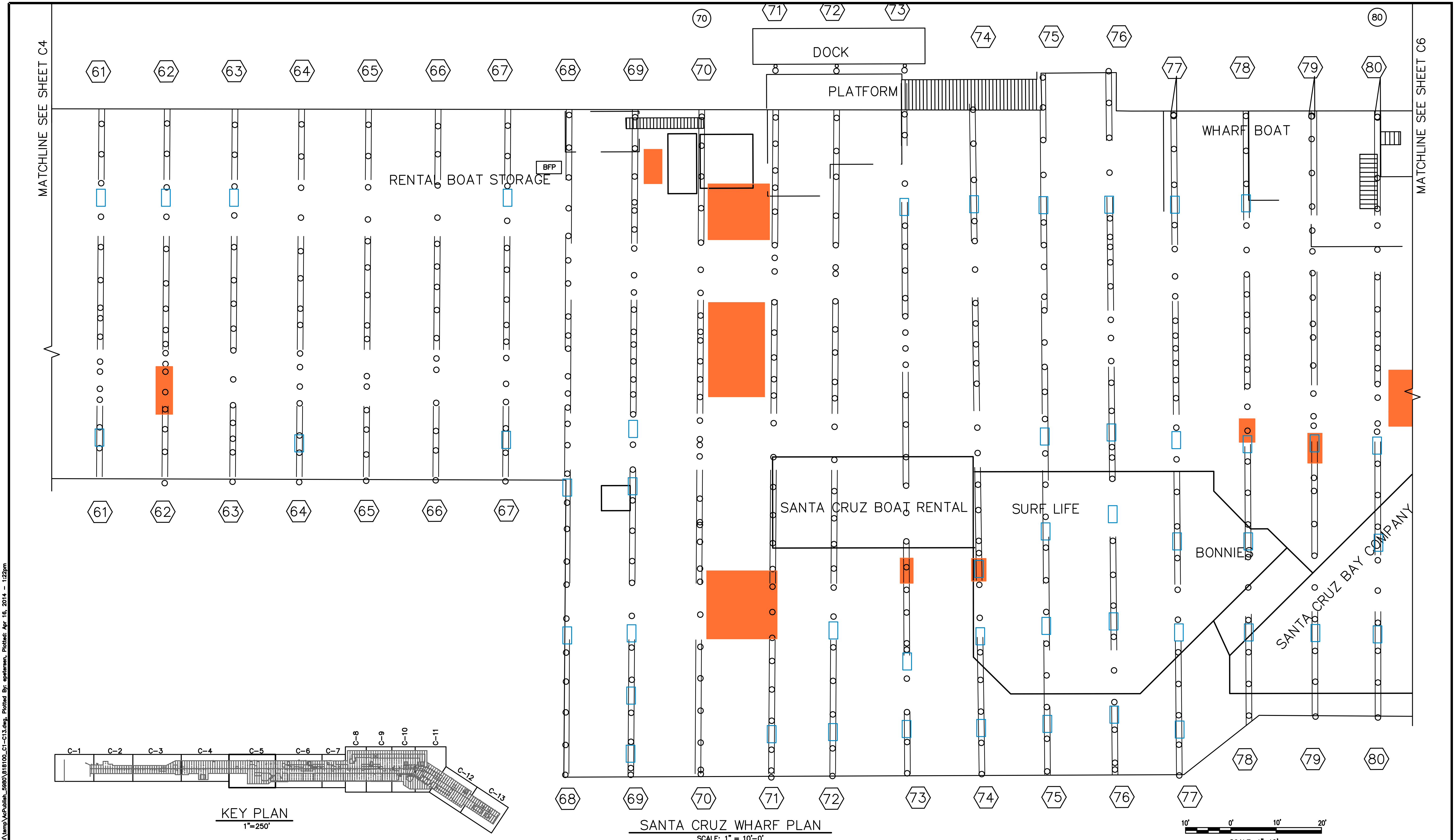

KEY PLAN
1"=250'

SANTA CRUZ WHARF PLAN
SCALE: 1" = 10'-0'

SCALE: 1"=10'

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DECK PLAN

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DRAWING #: #	DESIGN	BP	VAULT NO. #	
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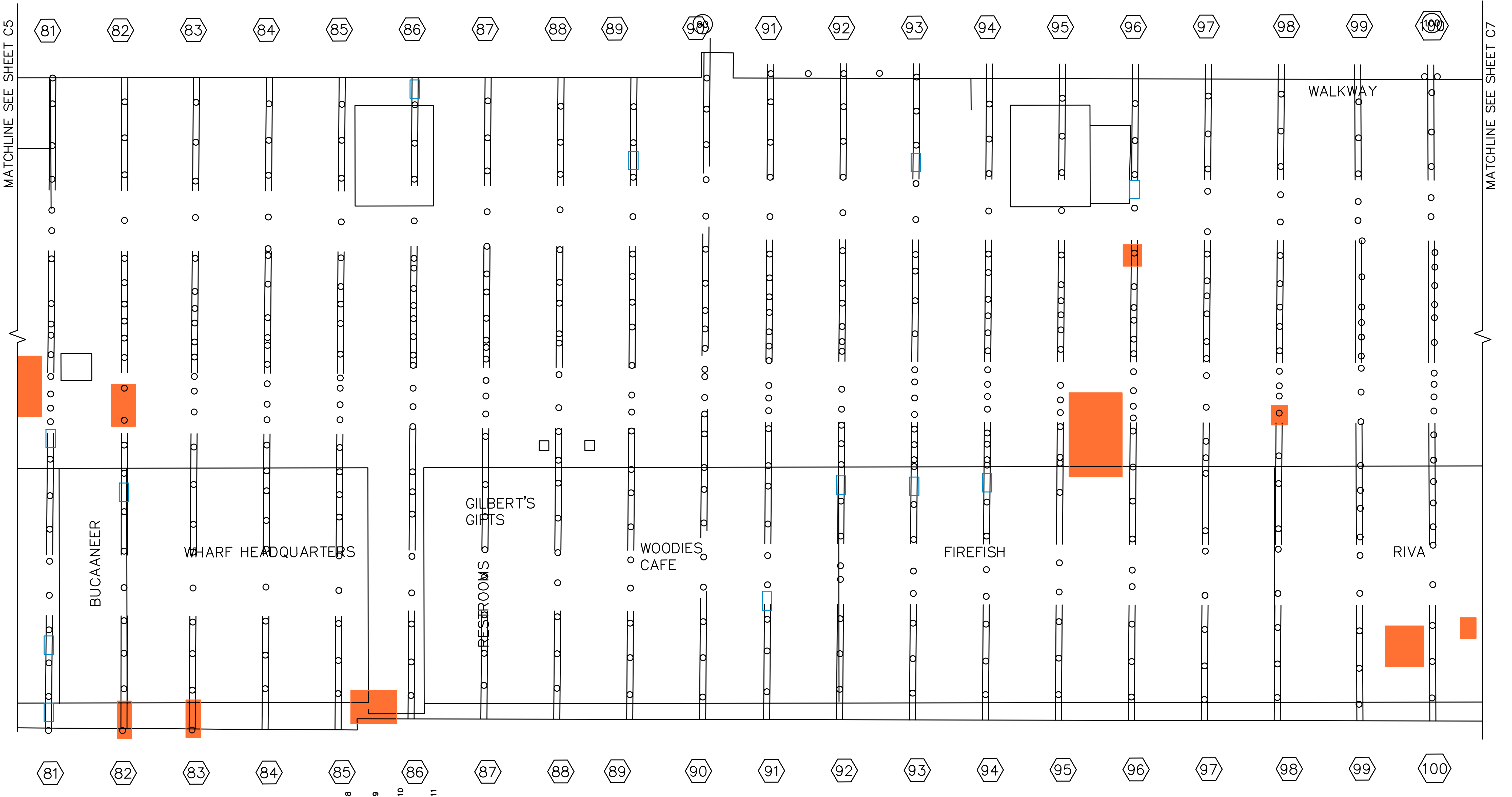
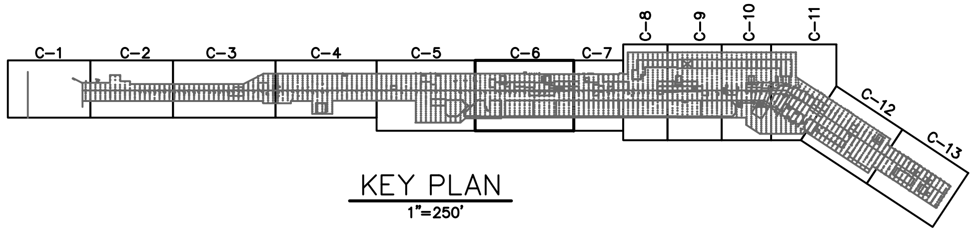


SANTA CRUZ WHARF PLAN

SCALE: 1" = 10'-0'



KEY PLAN

1"=250'

10' 0' 10' 20'

SCALE: 1"=10'

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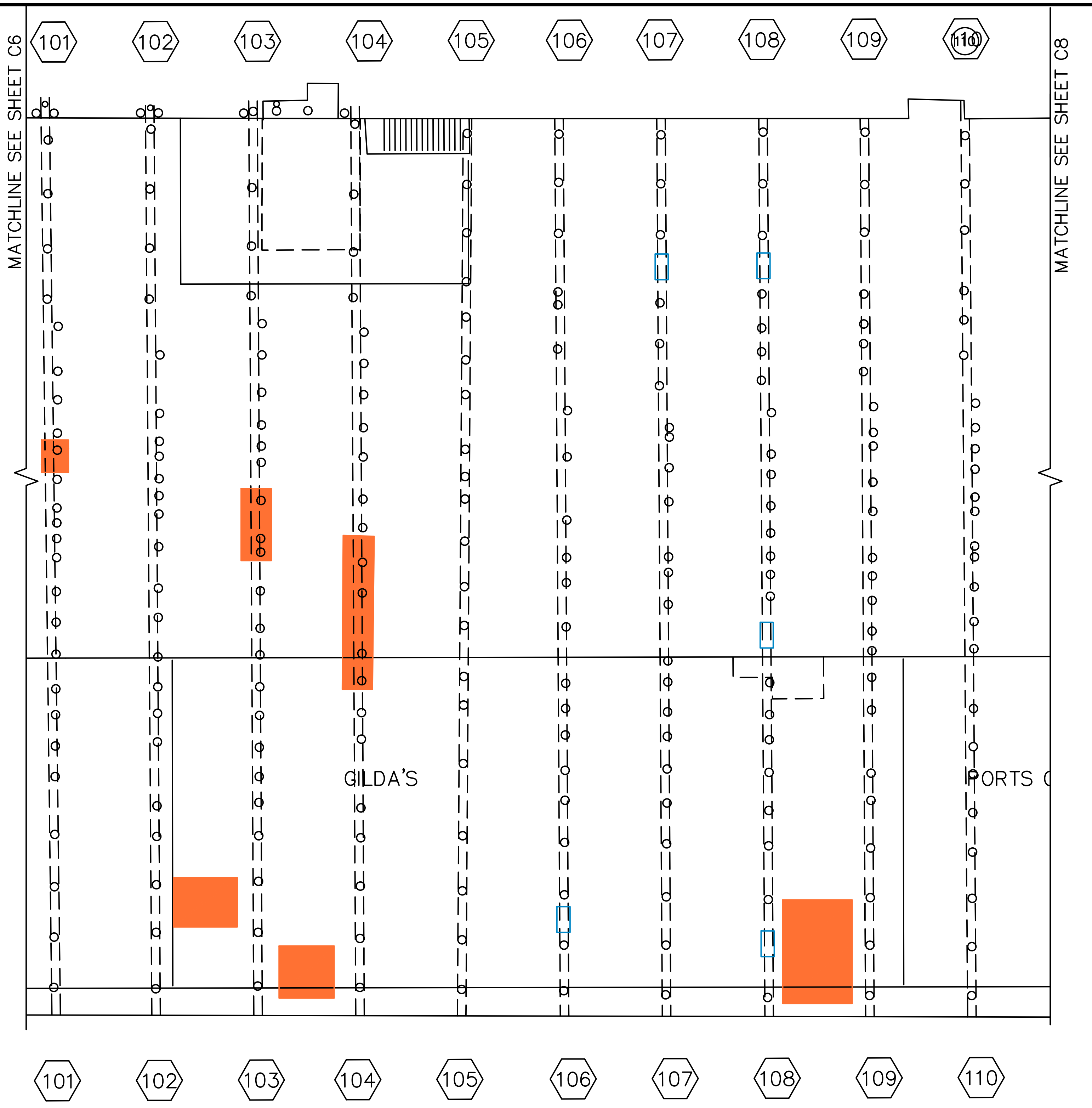

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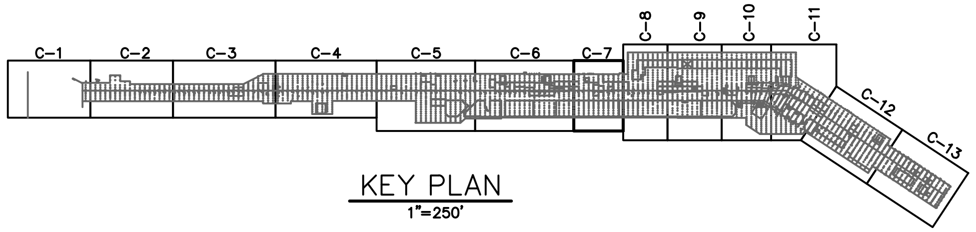
SCALE: 1" = 10'-0'



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SCALE: 1"=10'

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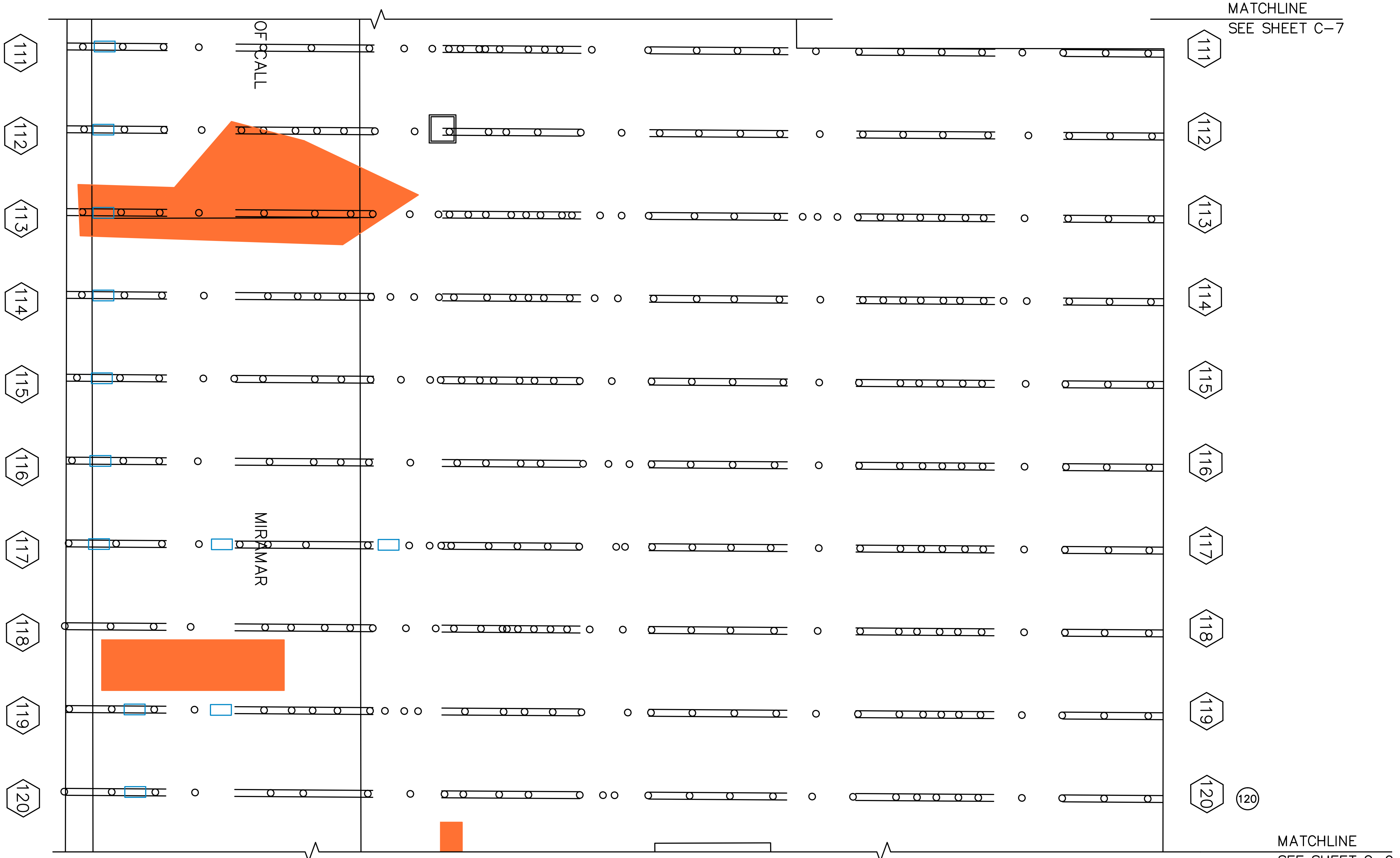
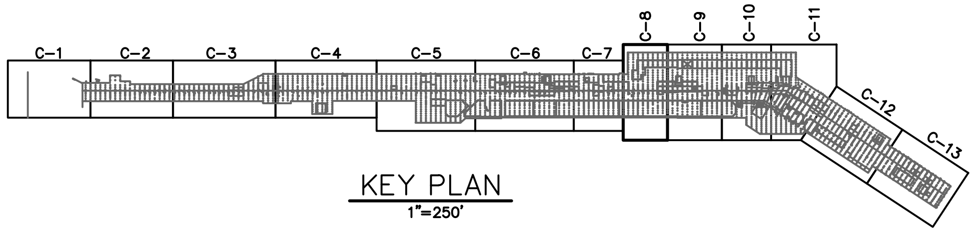


SANTA CRUZ WHARF PLAN
SCALE: 1" = 10'–0'



KEY PLAN
1"=250'

10' 0' 10' 20'
SCALE: 1"=10'

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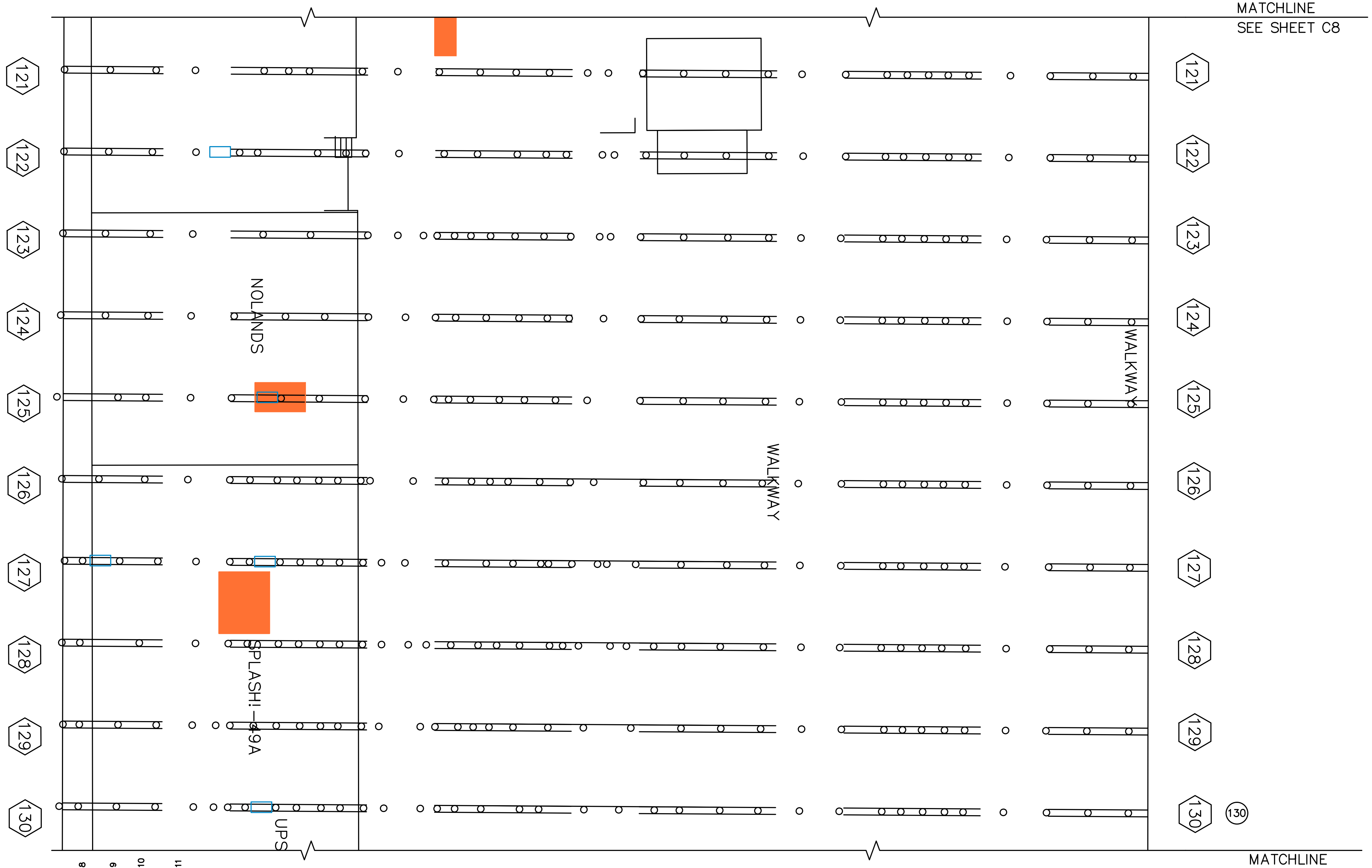
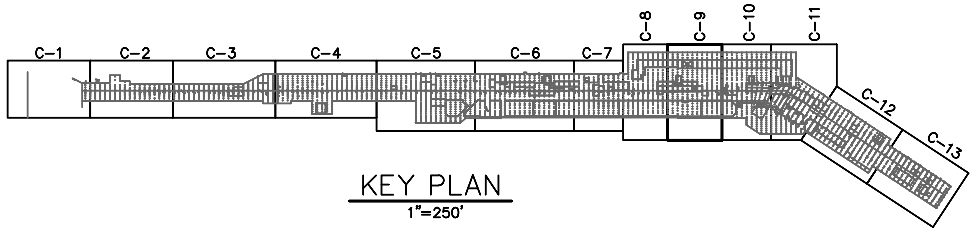


SANTA CRUZ WHARF PLAN

SCALE: 1" = 10'–0'



KEY PLAN

1"=250'

10' 0' 10' 20'

SCALE: 1"=10'

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DATE 6-15-11
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DESIGN BP
CHECKED BP

SCALE 1"=10'-0"
SHEET C9 OF -
VAULT NO.
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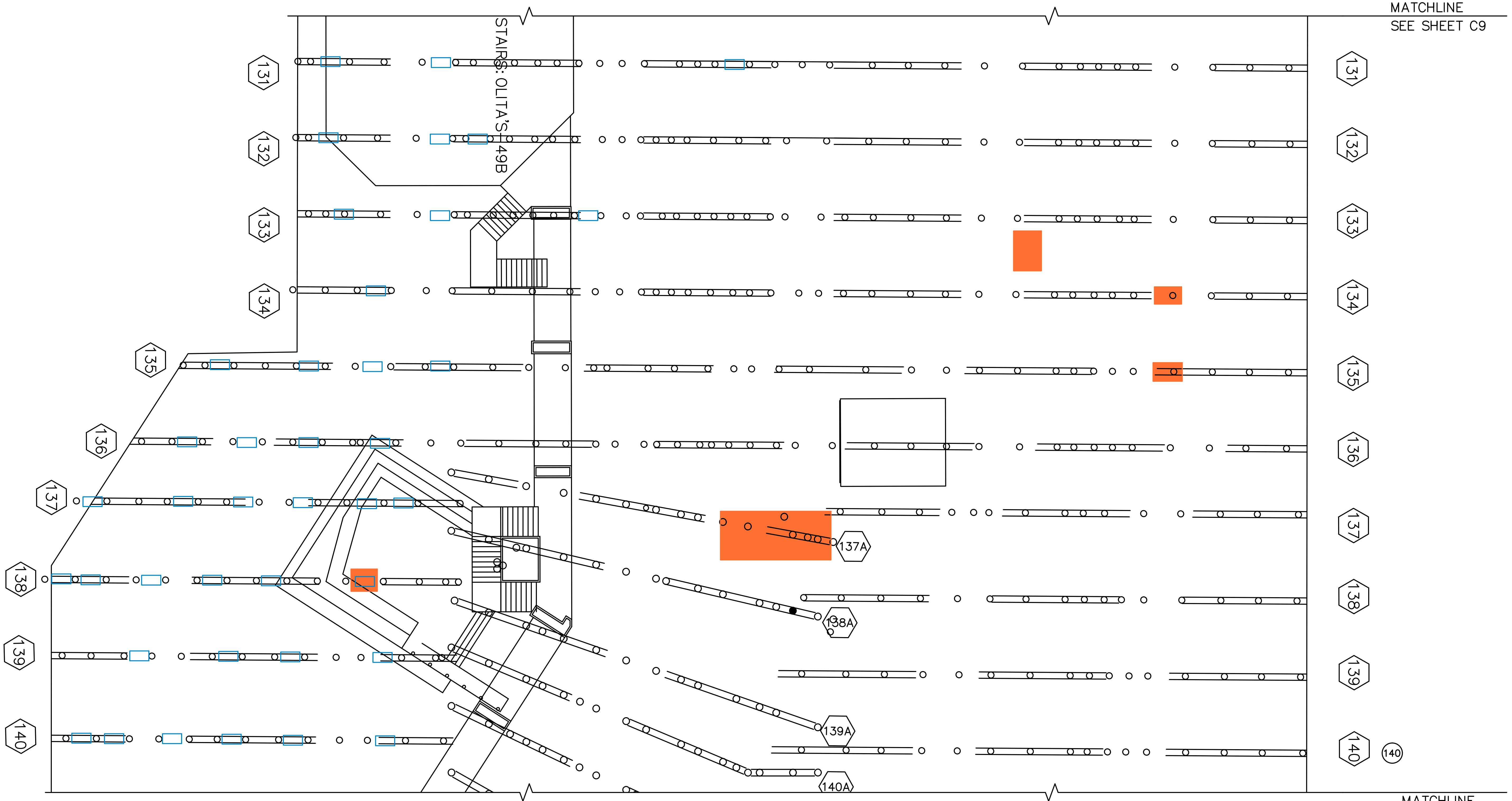
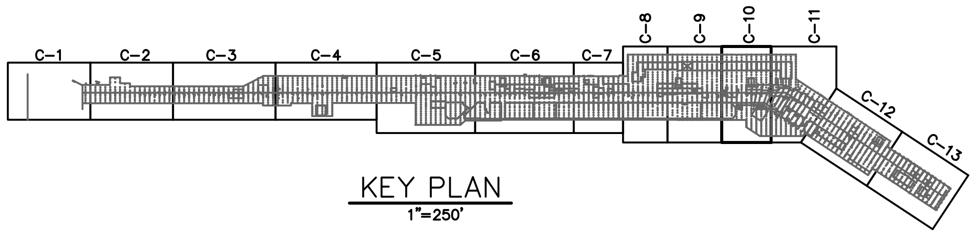


SANTA CRUZ WHARF PLAN

SCALE: 1" = 10'-0'



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SCALE: 1"=10'

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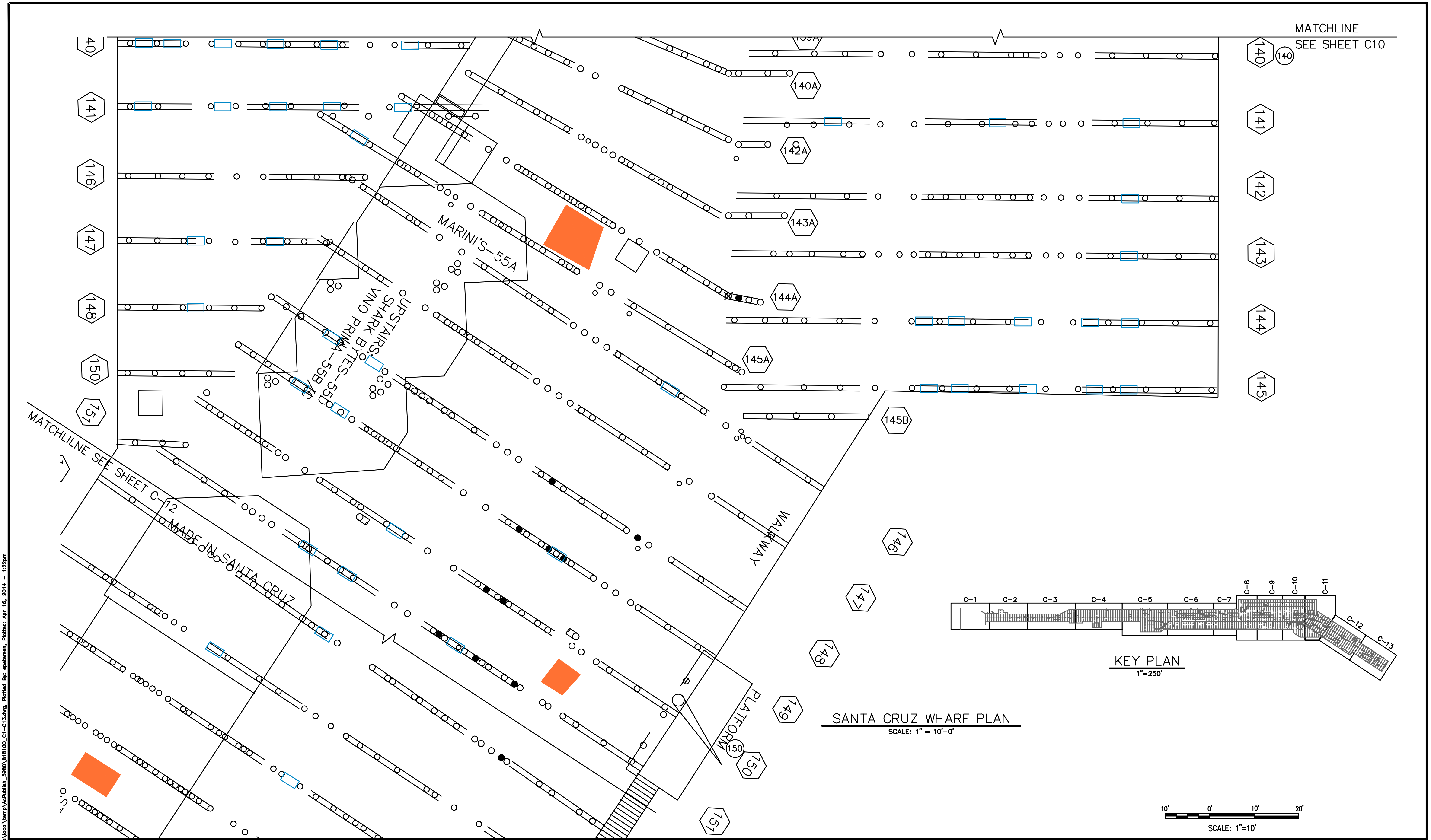

KEY PLAN
1"=250'

SANTA CRUZ WHARF PLAN
SCALE: 1" = 10'-0'

SCALE: 1"=10'

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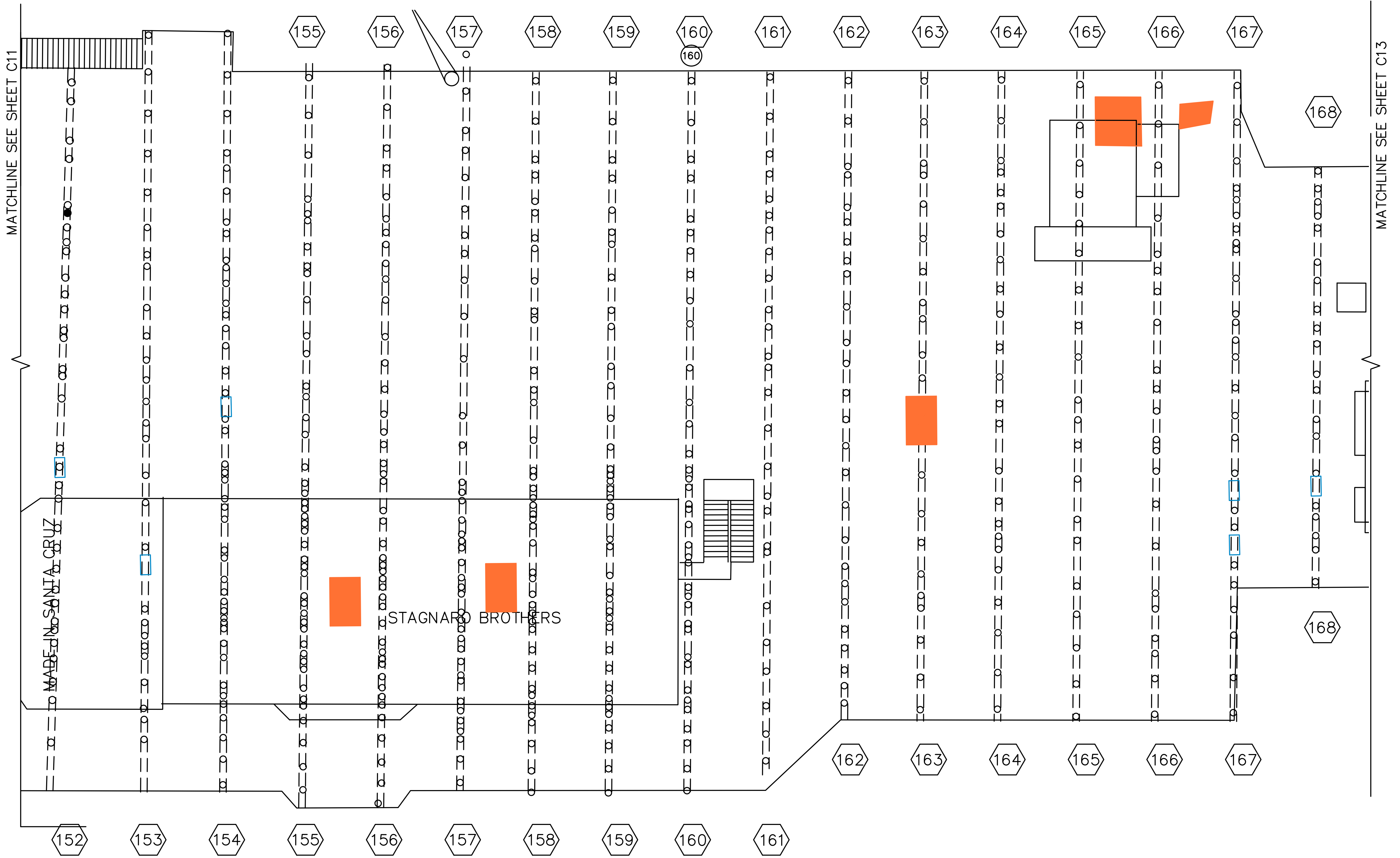

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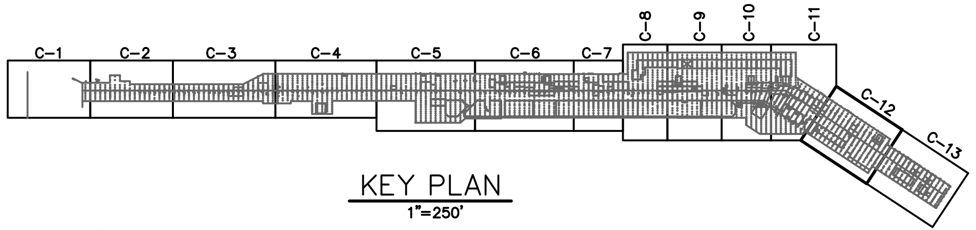

KEY PLAN

1"=250'

10' 0' 10' 20'

SCALE: 1"=10'

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CHECKED	BP

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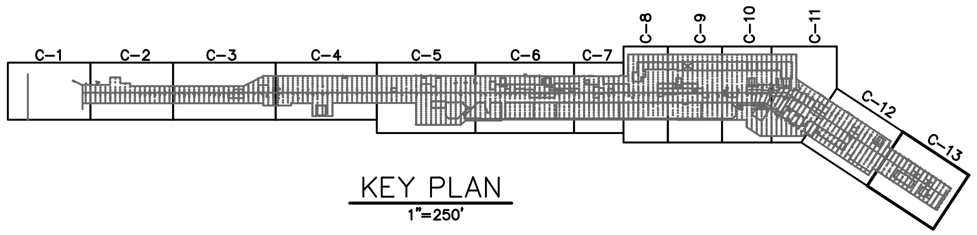
SHEET	C12 OF -
VAULT NO.	#

SANTA CRUZ WHARF PLAN

SCALE: 1" = 10'-0'



KEY PLAN

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10' 0' 10' 20'

SCALE: 1"=10'

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DESIGN	BP	VAULT NO.	#
CHECKED	BP		

Attachment 2C Calculations

Table of Contents

1 Wharf Structural Member Capacities

Design Parameters and Assumptions

Douglas Fir Larch (North) Select Structural

Assume has 19% moisture content for extended period of time (Ct)

Not exposed to high temperatures

Use NDS 2005, ASD

Member Dimensions

Bent Cap Dimensions 12x12
hc = 12 in height
wc = 12 in width
Stringer Dimensions 4x12
hs = 12 in height
ws = 4 in width
Stringer Dimensions 6x12
hs = 12 in height
ws = 6 in width
Decking Dimensions 3x12
hd = 3 in height
wd = 12 in width
Pile Dimensionts 12" round
diameter = 12 in

Section Modulus, S
Bent Cap = 288 in^3
Stringer 4x12 = 96
Stringer 6x12 = 144 in^3
Decking = 18 in^3
Pile = 170 in^3 pi*d^3/32

Bending Stress

	NDS Section Reference		Table 4 or 6A	4.3.2, 6.3	4.3.3, 6.3	4.3.4, 6.3	6.3.5	4.3.5, 6.3	4.3.6, 6.3	4.3.7, 6.3	4.3.8, 6.3	4.3.9, 6.3	6.3.11
Structural Member	**Allowable Stress**		Nominal Stress	Load Duration Factor	Wet Service Factor	Temperature Factor	Untreated Factor	Beam Stability Factor	Size Factor	Flat Use Factor	Incising Factor	Repetitive Member Factor	Single Pile Factor
	Fb' (psi)	=	Fb (psi)	* C_D	* C_m	* C_t	*C_u	* C_L	* C_F	* C_{fu}	* C_i	* C_r	* C_{sp}
Cap, 12x12	**1600**	=	1600	1	1	1	-	1	1	1	1	1	-
Stringer, 4x12	**1452**	=	1350	1	0.85	1	-	1	1.1	1	1	1.15	-
Stringer, 6x12	**1600**	=	1600	1	1	1	-	1	1	1	1	1	-
Decking, 3x12	**1768**	=	2000	1	0.85	1	-	1	1.04	1	1	1	-
Pile 12" diam.	**1887**	=	2450	1	1	1	1	1	1	1	1	1	0.77

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Shear Stress

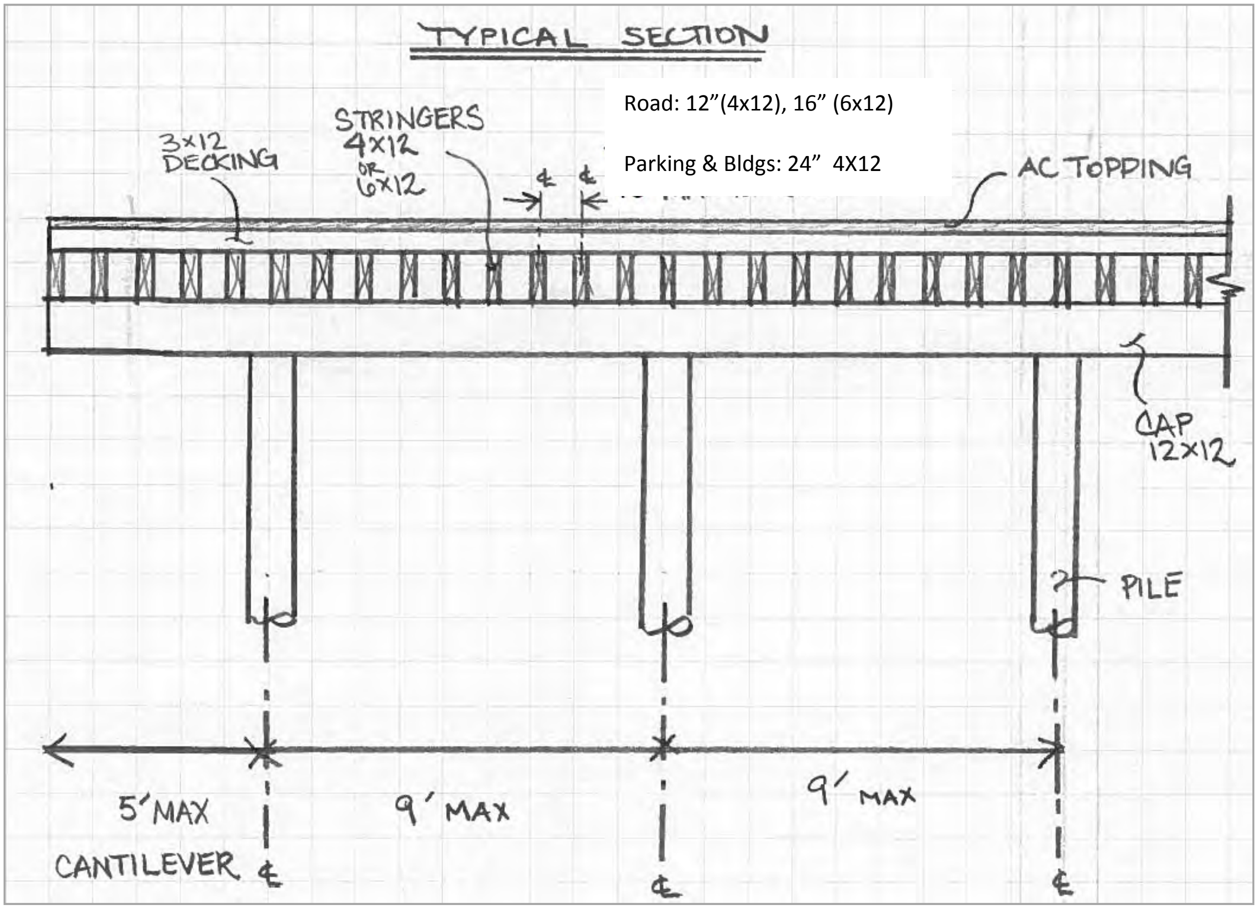
Structural Member	NDS Section Reference		Table 4 or 6A	4.3.2, 6.3	4.3.3, 6.3	4.3.4, 6.3	6.3.5	4.3.5, 6.3	4.3.6, 6.3	4.3.7, 6.3	4.3.8, 6.3	4.3.9, 6.3	6.3.11
	Allowable Stress		Nominal Stress	Load Duration Factor	Wet Service Factor	Temperature Factor	Untreated Factor	Beam Stability Factor	Size Factor	Flat Use Factor	Incising Factor	Repetitive Member Factor	Single Pile Factor
	Fv' (psi)	=	Fv (psi)	* C_D	* C_m	* C_t	*C_u	-	-	-	* C_i	-	* C_{sp}
Cap, 12x12	**170**	=	170	1	1	1	-	-	-	-	1	1	-
Stringer, 4x12	**175**	=	180	1	0.97	1	-	-	-	-	1	1	-
Stringer, 6x12	**170**	=	170	1	1	1	-	-	-	-	1	1	-
Decking, 3x12	**175**	=	180	1	0.97	1	-	-	-	-	1	1	-
Pile 12" diam.	**115**	=	115	1	-	1	1	-	-	-	-	-	-

Axial Stress

Structural Member	NDS Section Reference		Table 4 or 6A	4.3.2, 6.3	4.3.3, 6.3	4.3.4, 6.3	6.3.5	4.3.5, 6.3	4.3.6, 6.3	4.3.7, 6.3	4.3.8, 6.3	4.3.9, 6.3	6.3.11	4.3.10	6.3.9	3.6	3.7.1	3.7.1	Table 4 or 6A	3.7.1	3.7.1	3.7.1
	Allowable Stress		Nominal Stress	Load Duration Factor	Wet Service Factor	Temperature Factor	Untreated Factor	Beam Stability Factor	Size Factor	Flat Use Factor	Incising Factor	Repetitive Member Factor	Single Pile Factor	Column Stability Factor	Critical Section Factor	Length	Ke, 1.0= pinned-pinned, 0.7 = fixed btm-pinned	L/d * Ke	Emin	Fc* (same as Fc', but not mult. by Cp)	$F_{CE} = 0.822Emin'/(L/d*Ke)^2$	c = 0.8 for sawn lumber, 0.85 for poles and piles, 0.9 for structural lued laminated or composite
	Fc' (psi)	=	Fc (psi)	* C_D	* C_m	* C_t	*C_u	-	* C_F	-	* C_i	-	* C_{sp}	* C_p	* C_{cs}	ft	-	-	psi	-	-	-
Stringer, 6x12 COMP	**409**	=	1100	1	0.91	1	-	-	1	-	1	-	-	0.41	-	16	1	32.0	580000	1001	466	0.8
Stringer, 6x12 TENSION	**950**	=	950	1	1	1	-	-	1	-	1	-	-		-	-	-	-	-	-	-	-
Pile 12" diam.	**363**	=	1250	1	1	1	1	-	1	-	1	-	0.8	0.36	1.00	58	0.7	40.6	790000	1000	394	0.85

Structural Member	S, Section Modulus	Area (b*d)	Allowable Bending Moment	Allowable Shear Force	Allowable Axial Force	
			M = Fb' * S, *Eq 3.3-1*	V = Fv' * 2/3 * b *d, *Eq 3.4-2*	P = Fc' * A	
	in^3	in^2	lb-ft	lb	Compression (lb)	Tension (lb)
Cap, 12x12	288	144	38400	16320		
Double Cap, 12x12	576	288	76800	32640		
Stringer, 4x12	96	48	11613	5587		
Stringer, 6x12	144	72	19200	8160	29452	68400
Decking, 3x12	18	36	2652	4190		
Pile 12" diam.*	170	113	26670	9755	41048	

2 Vertical Load Analysis Under Roadway and New Promenade

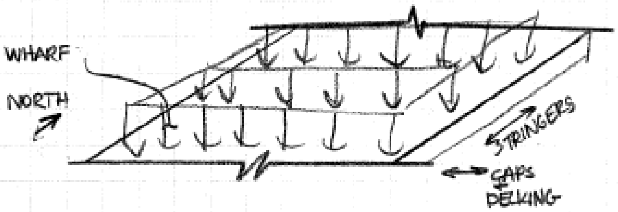


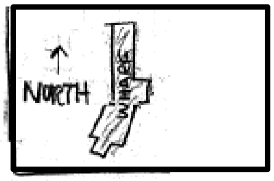
Road: 12"(4x12), 16" (6x12)

Parking & Bldgs: 24" 4X12

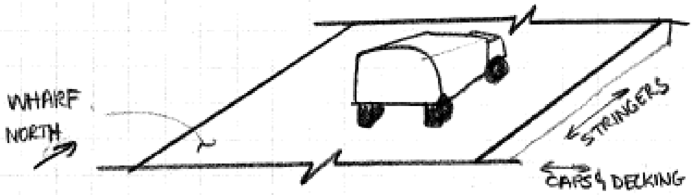
SUMMARY OF CASES

CASE 1 - Distributed Loads = 100 psf

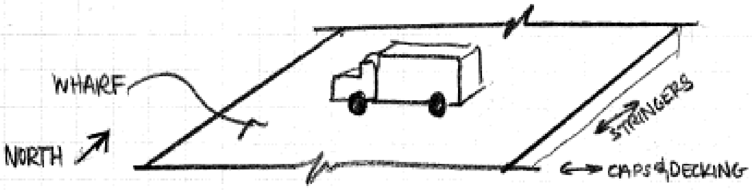




CASE 2A - TRUCK LOAD // TO ROAD (TYPICAL)



CASE 2B - TRUCK LOAD ⊥ TO ROAD (ONLY AT TURNAROUND & EAST PARKING)



v

2.1 Demand vs. Capacity – Vertical Loads

A summary of the results is presented here. The equations are shown on the subsequent pages.

For 4x12 stringers at 16” on center:

	Shear			Moment		
	Demand	Capacity*	DCR	Demand	Capacity*	DCR
	lb	lb	-	lb-ft	lb-ft	-
Case 1 - Distributed Loads						
Decking	80	4190	0.02	17	2652	0.01
Stringer	862	5587	0.15	4041	11613	0.35
Cap	11810	16320	0.72	22143	38400	0.58
Case 2A - Point Loads (Truck Axle parallel to Cap)						
Decking **	4684	20114	0.23	7063	12730	0.55
Stringer	3667	8940	0.41	14822	18580	0.80
Cap	18516	26112	0.71	32734	61440	0.53
Case 2B - Point Loads (Truck Axle parallel to Stringers)						
Decking **	5945	20114	0.30	7975	12730	0.63
Stringer	5337	8940	0.60	18628	18580	1.00
Cap	Case 2A governs over 2B for cap					

*Increase capacity by factor of 1.6 for Case 2 (truck loads). This is due to an increase in the load duration factor, C_D, from 1 to 1.6, based on NDS 2.3.2, considering a ten minute load duration

Adjusted C_D for Case 2: 1.6

** Values for Demand are from SAP analysis. Capacities are multiplied by 3 deck boards

For 6x12 stringers at 18” on center:

	Shear			Moment		
	Demand	Capacity*	DCR	Demand	Capacity*	DCR
	lb	lb	-	lb-ft	lb-ft	-
Case 1 - Distributed Loads						
Decking	106	4190	0.03	30	2652	0.01
Stringer	1160	8160	0.14	5436	19200	0.28
Cap	11918	16320	0.73	22346	38400	0.58
Case 2A - Point Loads (Truck Axle parallel to Cap)						
Decking **	6728	20114	0.33	8225	12730	0.65
Stringer	6038	13056	0.46	24382	30720	0.79
Cap	22252	26112	0.85	38803	61440	0.63
Case 2B - Point Loads (Truck Axle parallel to Stringers)						
Decking **	7298	20114	0.36	9790	12730	0.77
Stringer	8833	13056	0.68	30608	30720	1.00
Cap	Case 2A governs over 2B for cap					

*Increase capacity by factor of 1.6 for Case 2 (truck loads). This is due to an increase in the load duration factor, C_D, from 1 to 1.6, based on NDS 2.3.2, considering a ten minute load duration

Adjusted C_D for Case 2:	1.6

** Values for Demand are from SAP analysis. Capacities are multiplied by 3 deck boards

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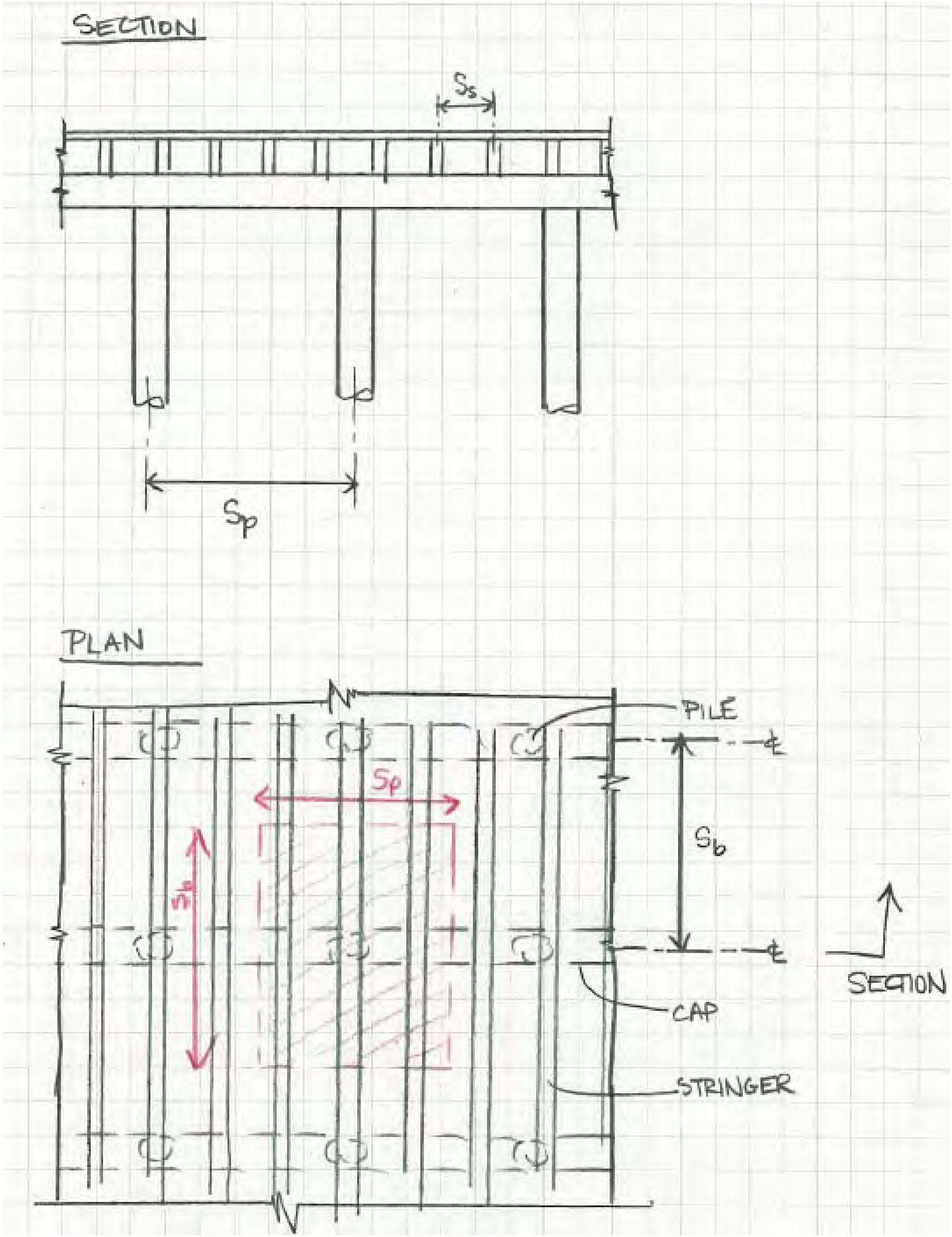
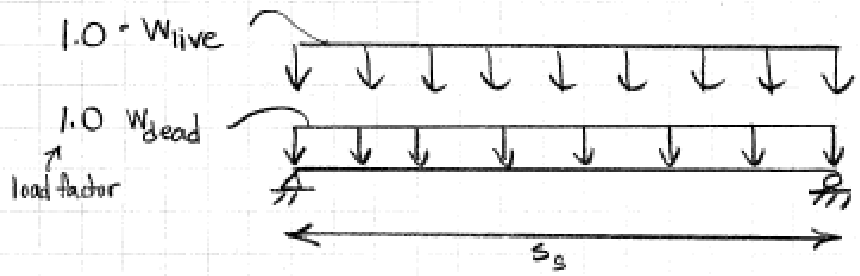
LABELLING

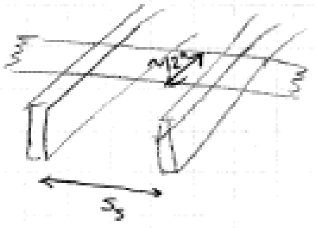


2.1.1 Case 1 – Distributed Loading

CASE 1 – DECKING DEMANDS

CASE 1 – DISTRIBUTED LOADS ON DECKING





$w_{live} = 100\,psf \cdot w_d$ (CBC 2010, Table 1607.1) For yards, terraces, pedestrians, stores, restaurants = lb/ft

↳ tributary width = 1' nominal

$w_{dead} = (w_{AC} \cdot h_t + w_{dougfir} \cdot h_d) \cdot w_d$ = lb/ft

↳ trib. width

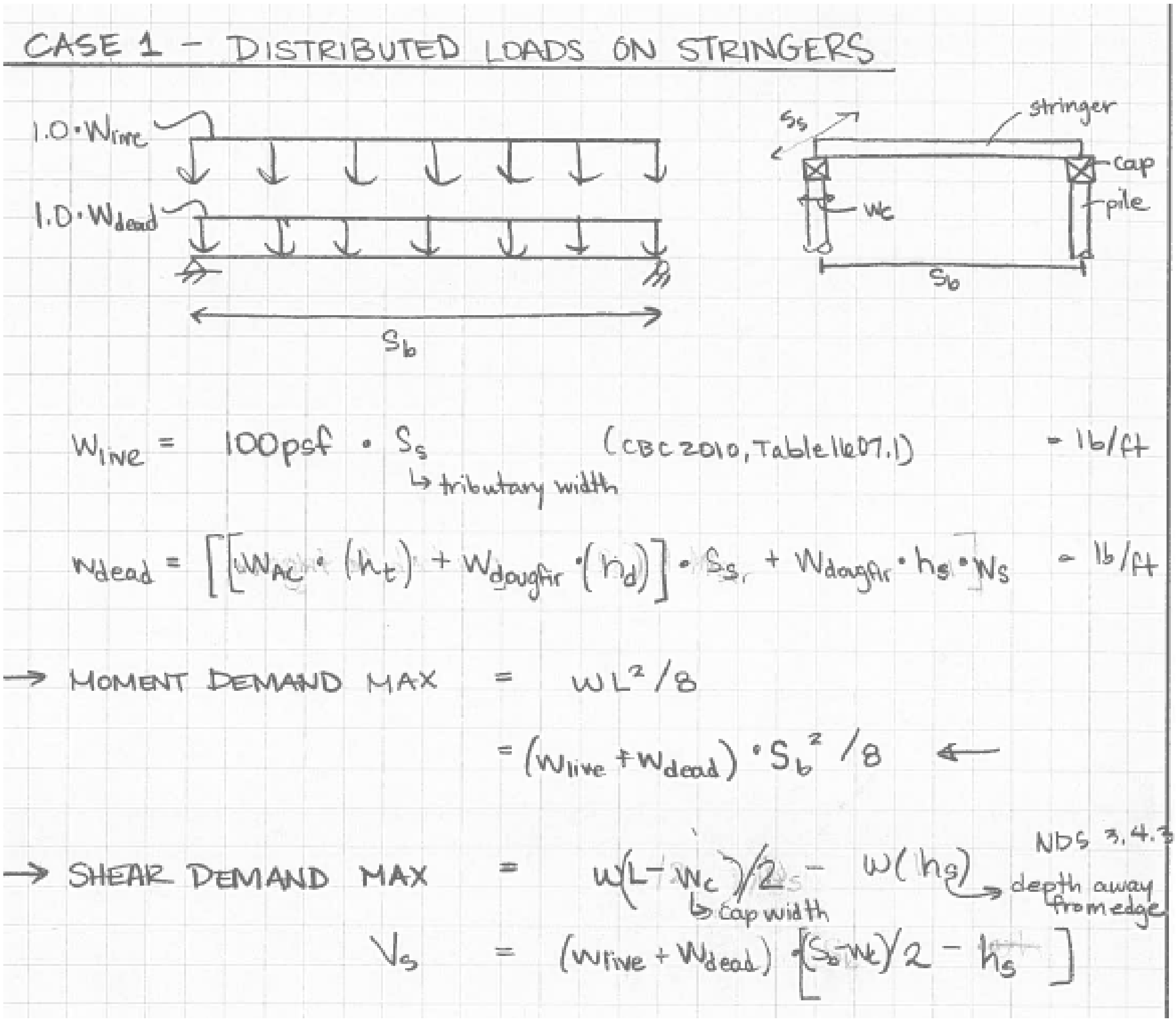
→ Moment Demand Max $= wL^2/8$ (conservatively assume simple span)

$= (w_{live} + w_{dead}) \cdot S_s^2 / 8$ ←

→ Shear Demand Max $= 0.6\,wL$ (for continuous beam, Table 3-23 in AISC 13th Ed.)

$= (w_{live} + w_{dead}) \cdot S_s \cdot 0.6$ ←

CASE 1 – STRINGER DEMANDS



CASE 1 - DISTRIBUTED LOADS ON STRINGERS

$W_{live} = 100psf \cdot S_s$ (CBC 2010, Table 1607.1) = lb/ft

↳ tributary width

$W_{dead} = \left[\left[W_{AC} \cdot (h_t) + W_{dougfir} \cdot (h_d)\right] \cdot S_s + W_{dougfir} \cdot h_s \cdot W_s\right]$ = lb/ft

→ MOMENT DEMAND MAX $= wL^2/8$

$= (W_{live} + W_{dead}) \cdot S_b^2 / 8$ ←

NDS 3.4.3

→ SHEAR DEMAND MAX $= w(L - W_c)/2 - w(h_s)$

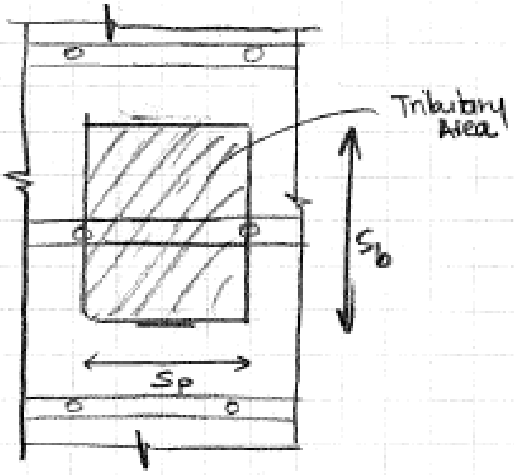
↳ cap width

→ depth away from edge

$V_s = (W_{live} + W_{dead}) \left[(S_b - W_c)/2 - h_s\right]$

Note: Stringers on wharf are simple span

CASE 1 – CAP DEMANDS



DEAD & LIVE DISTRIBUTED ON CAP

$$w_{\substack{DEAD\\CAP}} = \underline{w_{dougfir} \cdot h_c \cdot w_c}$$ CAP DEAD

$$+ \left(\underline{w_{dougfir} \cdot h_s \cdot w_s \cdot S_b/S_s}\right)$$ STRINGER DEAD

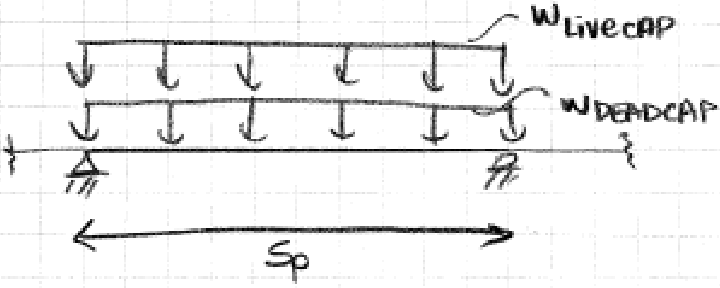
$$+ \left(\underline{w_{dougfir} \cdot h_d \cdot S_b}\right)$$ DECKING DEAD

$$+ \left(\underline{w_{AC} \cdot h_t \cdot S_b}\right)$$ AC DEAD

$$= w_{dougfir}\left(h_c W_c + h_s W_s \frac{S_b}{S_s} + h_d \cdot S_b\right) + W_{AC} \cdot h_t \cdot S_b$$

$$W_{\substack{LIVE\\CAP}} = w_{Live} \cdot S_b$$

CASE 1 - DISTRIBUTED LOADS ON CAPS



→ MOMENT DEMAND MAX $= wL^2/8$

$$= \frac{(w_{\text{LIVE CAP}} + w_{\text{DEAD CAP}}) \cdot S_p^2}{8}$$

conservatively assume simple span (see previous calcs for $w_{\text{Dead}} + w_{\text{Live}}$)

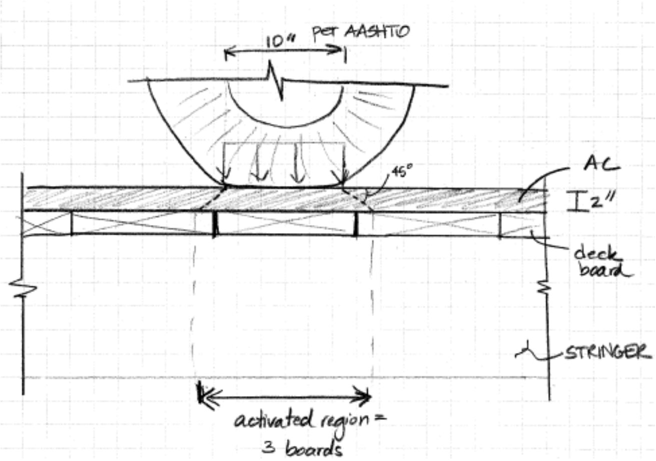
→ SHEAR DEMAND MAX $= 0.6wL$ (for continuous beam) AISC 13th Ed, Table 3-23

$$= (w_{\text{Live CAP}} + w_{\text{Dead CAP}}) \cdot 0.6 \cdot S_p$$

2.1.2 Case 2A – Point Loads, Truck Axle Parallel to Cap (Typical Case)

CASE 2A – DECKING DEMANDS

Decking demands are taken from a SAP model. The truck tire(s) load is spread out onto 3 deck boards, as this is a realistic distribution.



Note on Deck Demand calcs :

It was noticed that "rev1" (values used for results in Jan23 memo) didn't use stiffnesses from 4"x12" + 6"x12" members. (They were left as 3.5x11.5, 5.5x11.5) "Rev 2" check was done to adjust for larger members + higher stiffness. It was found that "rev1" was conservative, so these calcs use "rev1" to be consistent w/ memo
ETP 2/3/14

"Rev 2" values are in green

changes	rev1	rev 2
4x12	3.5x11.5	4x12
6x12	5.5x11.5	6x12
deck	3.5" thick	3" thick

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SPRING STIFFNESS OF STRINGER - For vehicle // to roadway @ midspan

4x12 stringer, E = 1900000 psi

6x12 stringer, E = 1600000 psi

$L = 14' + 3'' + 3' = 14.5'$ (NDS 3.2.1, clear span + 1/2 bearing length each end)

↙ conservatively use values for 3.5x11.5 + 5.5x11.5

$I_{4x12} = 444 \text{ in}^4$ 576 }

$I_{6x12} = 697 \text{ in}^4$ 864 } values used for rev2 comparison check

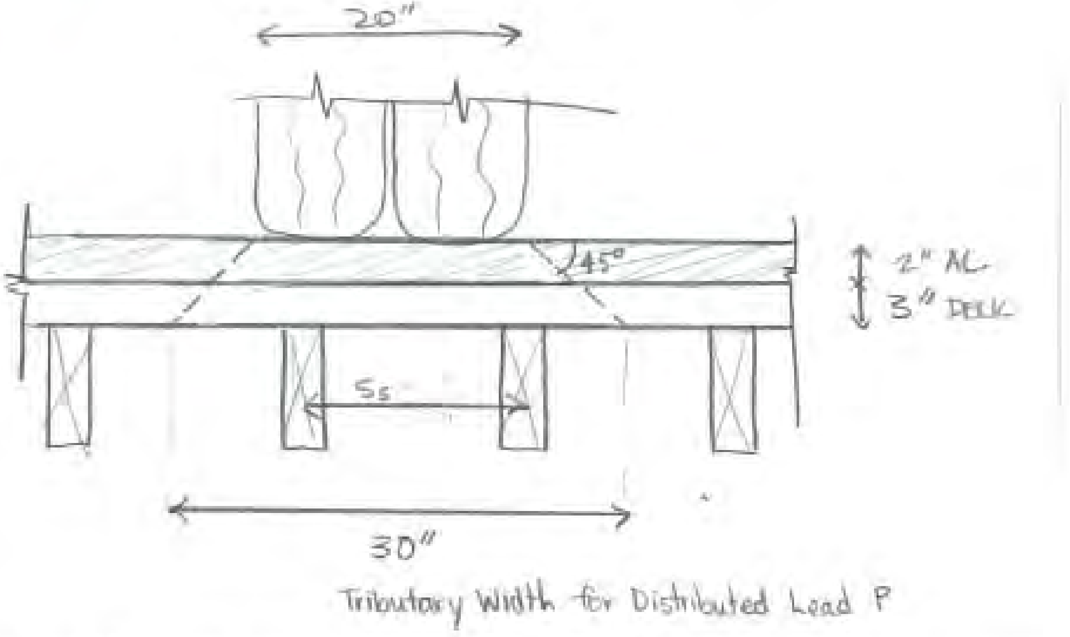
$K = P/\Delta = 48EI/L^3$ (P is a unit load)

values used for rev2

$K_{4x12} = 48 \cdot 1900000^{psi} \cdot 576^{in^4} / (12 \cdot 14.5^{in})^3 = 7.7 \text{ K/in}$ 10

$K_{6x12} = 48 \cdot 1600000^{psi} \cdot 864^{in^4} / (12 \cdot 14.5^{in})^3 = 10.2 \text{ K/in}$ 14

Convert Point Load P to a Distributed Load

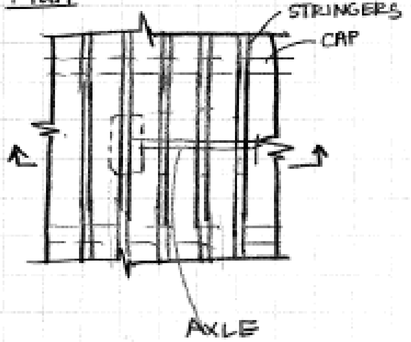


$P = 1 \text{ Kip} \rightarrow w = 1 \text{ k}/30 \text{ in} = \boxed{33.3 \text{ lb/in}}$ (across 3 deck boards)

CASE 2A – STRINGER DEMANDS

CASE 2A – POINT LOADS ON STRINGER (Truck // to CAP)

Plan:

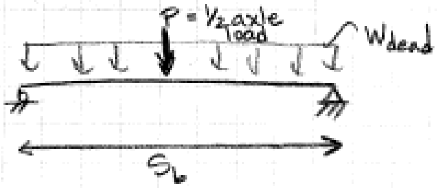


n = % of P on 1 stringer

Use Caltrans Bridge Design Specifications (2004) Table 3.23.1 to get n:

n = S/4, where S = stringer spacing

For	Ss = 12,	n = 25%
	Ss = 16,	n = 34%
	Ss = 18,	n = 38%



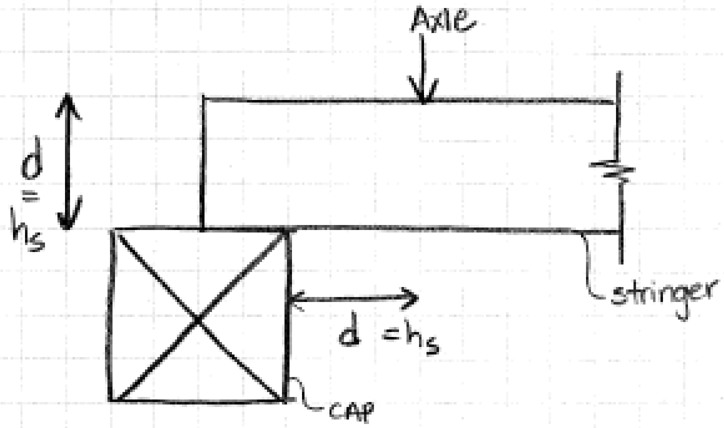
→ MOMENT DEMAND MAX = $\underbrace{wL^2/8}_{\text{dead}} + \underbrace{PL/4}_{\text{truck live}}$

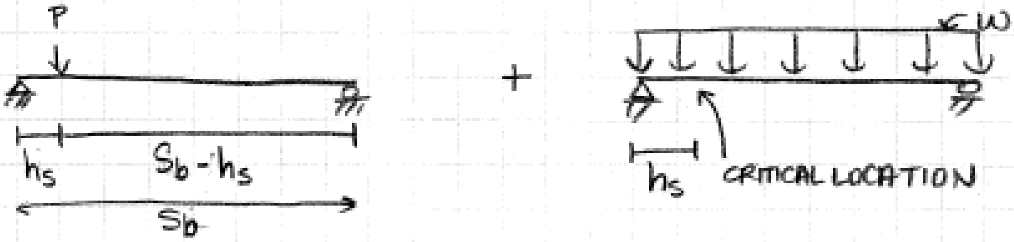
$= w_{\substack{\text{dead}\\\text{str.}}} \cdot S_b^2/8 + (P \cdot S_b/4) * n \leftarrow$

See Case 1

→ MAX SHEAR

MAXIMUM SHEAR IS WHEN LOAD IS "D" AWAY FROM FACE OF SUPPORT:

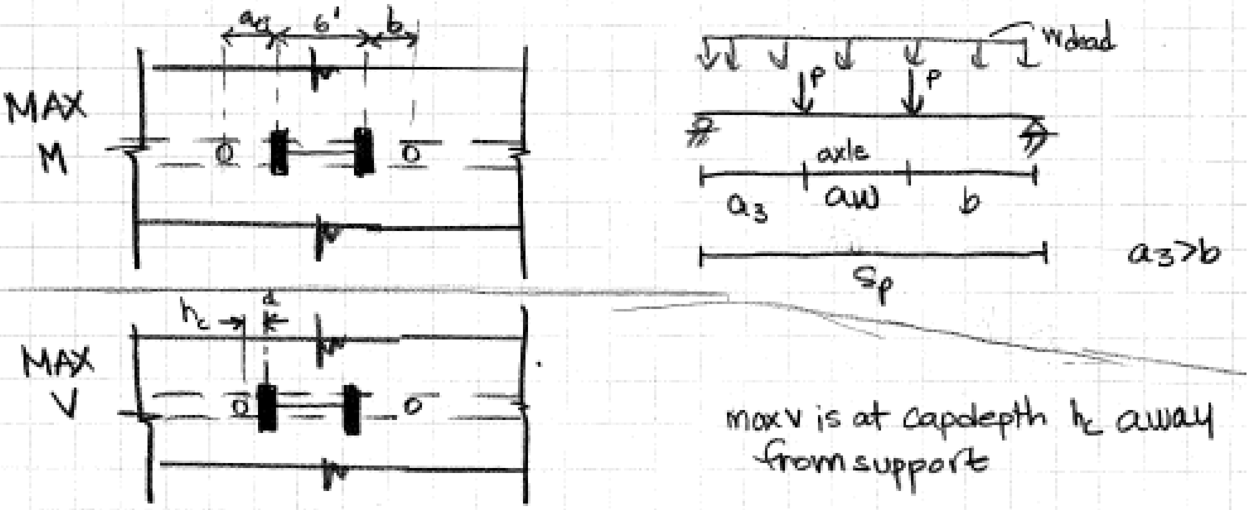




$$V = P \cdot (S_b - h_s)/S_b \cdot n \quad + \quad w\left(\frac{S_b}{2} - h_s\right)$$

CASE 2A – CAP DEMANDS

CASE 2A - POINT LOADS ON CAPS (Truck 11 to CAP)



max V is at capdepth h_c away from support

→ MOMENT DEMAND MAX

$$= \frac{P a_3}{L}(L - a_3 + b) + \frac{w_{dead}\, a_3}{2}(L - a_3)$$

$$= \frac{P \cdot a_3}{s_p}(2s_p - 2a_3 - a_w) + w_{dead_{CAP}}(s_p^2)/8$$

(see prev. calc)

→ SHEAR DEMAND MAX

$$= P/_L(L - a + b) + w_{dead}\left(\frac{L}{2} - a\right)$$

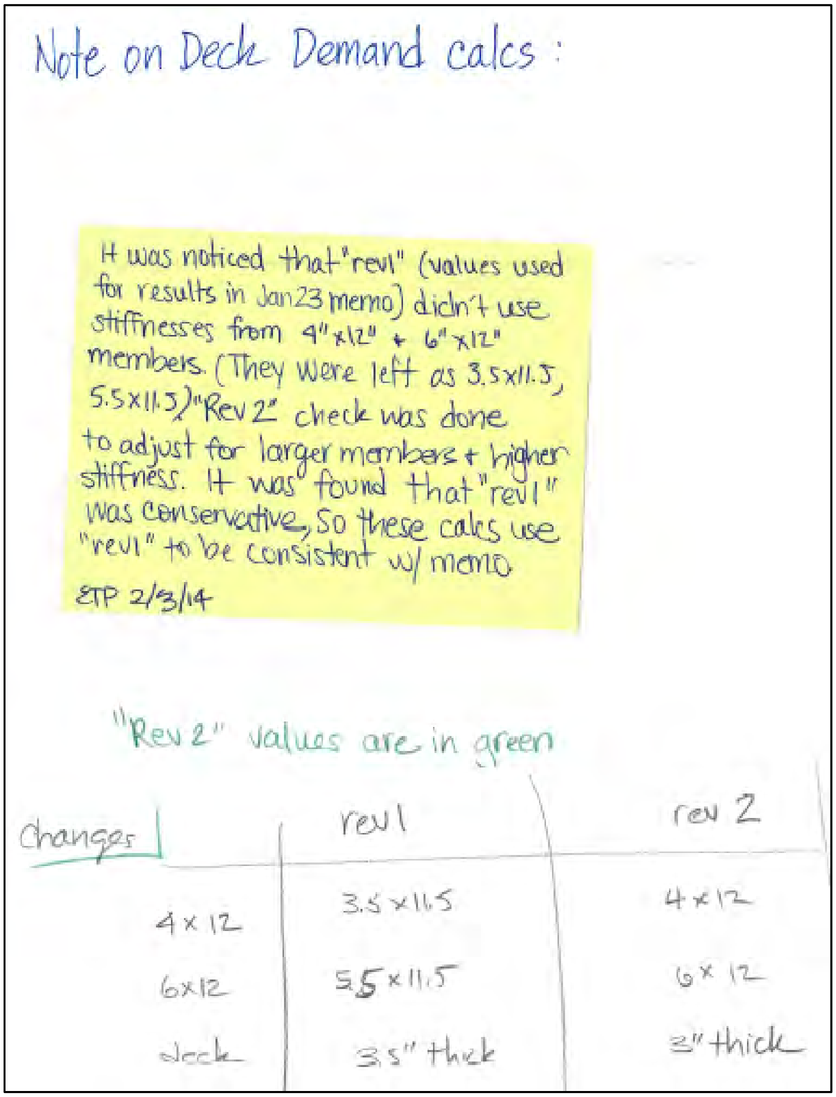
$$= \frac{P}{s_p}\left(s_p - h_c + \overset{(b)}{[s_p - a_w - h_c]}\right) + w_{dead_{CAP}} \cdot \left(\frac{s_p}{2} - h_c\right)$$

(see prev. calc.)

2.1.3 Case 2B - Point Loads, Truck Axle Perpendicular to Cap (Only at Turnaround and East Parking Lot Roadway)

CASE 2B – DECKING DEMANDS

Decking demands are taken from a SAP model. The truck tire(s) load is spread out onto 3 deck boards, as this is a realistic distribution.



Note on Deck Demand calcs:

It was noticed that "rev1" (values used for results in Jan23 memo) didn't use stiffnesses from 4"x12" + 6"x12" members. (They were left as 3.5x11.5, 5.5x11.5) "Rev 2" check was done to adjust for larger members + higher stiffness. It was found that "rev1" was conservative, so these calcs use "rev1" to be consistent w/ memo.

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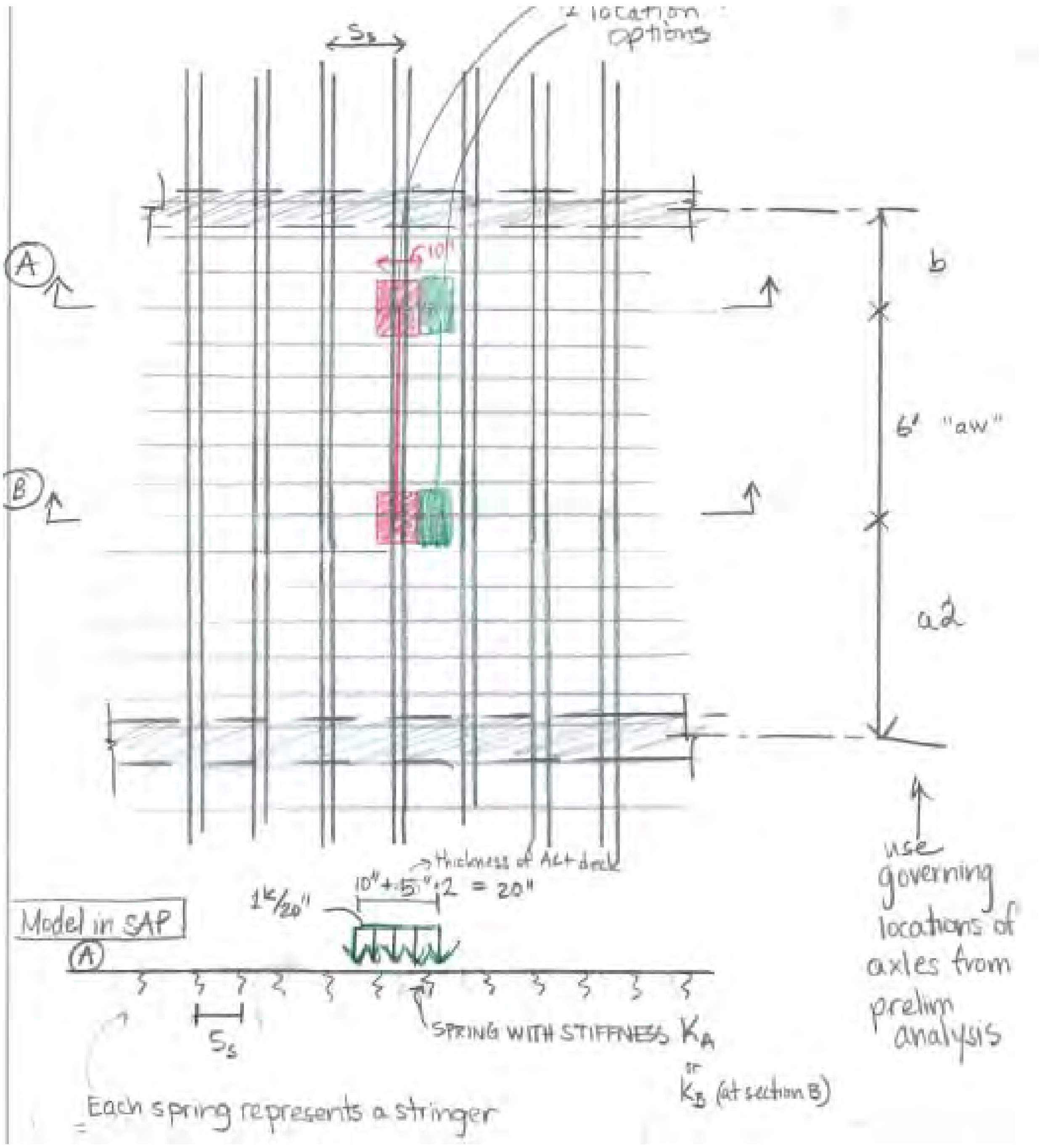
"Rev 2" values are in green

changes	rev1	rev 2
4x12	3.5x11.5	4x12
6x12	5.5x11.5	6x12
deck	3.5" thick	3" thick



2 location options
S_s
10"
A
B
b
6' "aw"
a2
use governing locations of axles from prelim analysis
thickness of AC+deck
10"+5"·2 = 20"
1k/20"
Model in SAP
A
S_s
SPRING WITH STIFFNESS K_A or K_B (at section B)
Each spring represents a stringer

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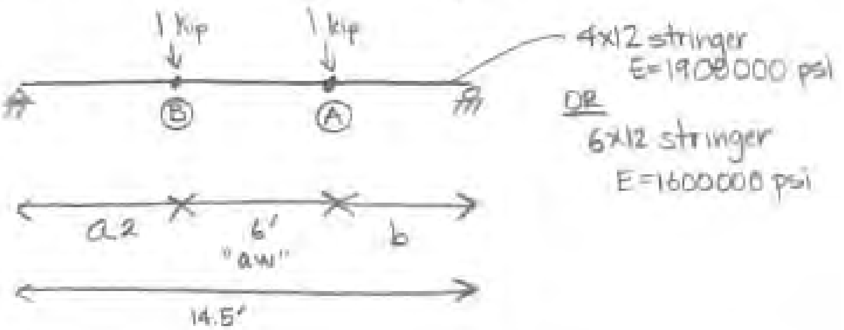
STRINGER SPRING STIFFNESS – For vehicle ⊥ to roadway

SAP MODEL



RESULTS

From SAP

K = 1kip/Δ

Stringer Size	a_2	b	Δ_B	Δ_A	K_B	K_A
4x12	6.2'	2.8'	0.183" 0.14"	0.095" 0.07	5.5 k/in 7.1	10.5 k/in 14.3
6x12	6.2'	2.8'	0.116" 0.094"	0.060" 0.05"	8.6 k/in 10.6	16.7 k/in 20
4x12	7'	1.5'	0.171" 0.13"	0.058" 0.05	5.8 k/in 7.7	17.2 k/in 20
6x12	7'	1.5'	0.109" 0.09"	0.037" 0.03"	9.2 k/in 11.1	27 k/in 33.3

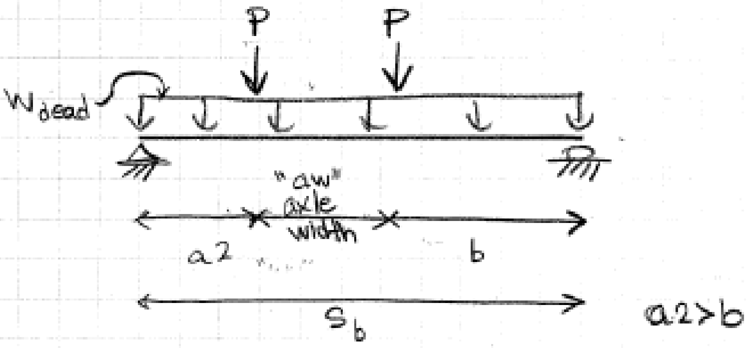
Green values are for rev2 comparison check

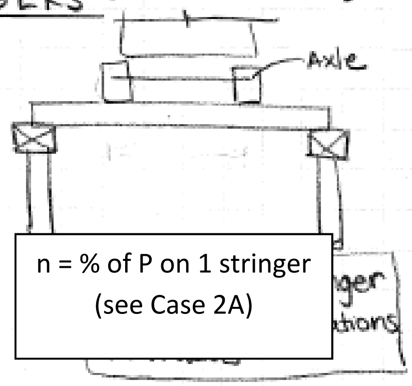
rev2 values

→ Less stiff springs govern for deck demands, because it will deflect more, causing higher moments.

CASE 2B – STRINGER DEMANDS

CASE 2B — POINT LOADS ON STRINGERS (Truck ⊥ to cap)





n = % of P on 1 stringer (see Case 2A)

$$\longrightarrow \text{Moment Demand Max} = \underset{\text{dead}}{wL^2/8} + \frac{Pa2}{L}(L - a2 + b)$$

$$= (w_{dead})\cdot S_b^2/8 + \frac{Pa2}{S_b}\left(S_b - a2 + [S_b - a2 - aw]\right)$$

$$= (w_{dead})\cdot S_b^2/8 + \left[\frac{Pa2}{S_b}(2S_b - 2a2 - a_w)\right]\cdot n \leftarrow$$

$\longrightarrow$ Shear Demand Max

max is when tire is stringer depth "h_s" away from face of support. $a_2 = \frac{w_c}{2} + h_s$

$$= P/L(L - a + b)\cdot n + w_{dead}\left(\frac{L}{2} - a\right)$$

$$= \frac{P}{S_b}\left(S_b - \left(\frac{w_c}{2} + h_s\right) + S_b - aw - \left(\frac{w_c}{2} + h_s\right)\right)\cdot n + w_{dead}\left(\frac{S_b}{2} - \left(\frac{w_c}{2} + h_s\right)\right)$$

$$= \frac{P}{S_b}\left(2S_b - aw - w_c - 2h_s\right)\cdot n + w_{dead}\left(\frac{S_b}{2} - \frac{w_c}{2} - h_s\right)$$

CASE 2B – CAP DEMANDS

Case 2A governs for Cap demands.

2.1.4 Design Check

Decking Demands:

CASE 2A - Decking Demands from SAP (unit 1 kip distrib load):

	V (lb)	M (lb-ft)
4x12@12	323	487
6x12@16	378	462

CASE 2B - Decking Demands from SAP (unit 1 kip distrib load):

	V (lb)	M (lb-ft)
4x12@12	420	644
6x12@16	410	550

Since these demands are for a unit axle load, these values will be multiplied by the axial load and then compared to the capacity for 3 deck boards.

Case 2B governs the analysis, however it was desired to show the capacity of the wharf for Case 2A only. The following calculations are first presented for CASE 2A only, and then CASE 2B governing.

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2.1.4.1 CASE 2A Only

Key
user input
user input to adjust
calculated formula

Design Parameters - General					
Sb	Bent Spacing	=	15	ft	
Sp	Pile Spacing	=	9	ft	
ww	wheel width	=	1.667	ft	AASHTO 2007, 3.6.1.2.5
aw	axle width	=	6	ft	
Design Parameters - 4x12 Stringers					
P	Truck Axle/2	=	18500	lb	Fire Truck
Ss	Stringer Spacing	=	1	ft	
a2	dist. to axle for stringers (Case 2B)	=	6	ft	a>b, yes Adjust these values to find max demand
a3	distance to axle for cap (Case 2A)	=	2	ft	a>b, yes
$n_{Case\ 2A}$	% of P in 1 stringer	=	0.25		based on Caltrans 2004 Bridge Design
$n_{Case\ 2B}$	% of P in 1 stringer	=	0.25		Specifications, Table 3.23.1 and SAP analysis
Design Parameters - 6x12 Stringers					
P	Truck Axle/2	=	21200	lb	Fire Truck
Ss	Stringer Spacing	=	1.333	ft	
a2	dist. to axle for stringers (Case 2B)	=	6.4	ft	a>b, yes Adjust these values to find max demand
a3	distance to axle for cap (Case 2A)	=	2	ft	a>b, yes
$n_{Case\ 2A}$	% of P in 1 stringer	=	0.34		based on Caltrans 2004 Bridge Design
$n_{Case\ 2B}$	% of P in 1 stringer	=	0.34		Specifications, Table 3.23.1 and SAP analysis
Wdougfir	Unit wt of doug fir	=	32	pcf	
Wac	unit weight of AC	=	150	pcf	
Live	live load	=	100	psf	*CBC 2010 Table 1607.1*
Bent Cap Properties		12x12			
hc	height	=	12	in	
wc	width	=	12	in	
Stringer Properties		4x12			
hs	height	=	12	in	
ws	width	=	4	in	
E	Modulus of Elasticity	=	1900000	psi	NDS Table 4A, Douglas Fir Larch (North)
I	Moment of Inertia	=	576	in^4	$ws*hs^3/12$
Stringer Properties		6x12			
hs	height	=	12	in	
ws	width	=	6	in	
E	Modulus of Elasticity	=	1600000	psi	NDS Table 4D, Douglas Fir Larch (North)
I	Moment of Inertia	=	864	in^4	$ws*hs^3/12$
Decking Properties		3x12			
hd	height	=	3	in	
wd	width	=	12	in	
Deck Topping Properties		AC			
ht	height	=	2	in	

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CASE 1 - DISTRIBUTED LOADING

		4x12 Stringers		6x12 Stringers		
Loads on Decking						
w_{live}	=	100	lb/ft	100	lb/ft	
w_{dead}	=	33	lb/ft	33	lb/ft	
Moment Demand	=	17	lb-ft	30	lb-ft	
Shear Demand	=	80	lb	106	lb	
Loads on Stringers						
w_{live}	=	100	lb/ft	133	lb/ft	
w_{dead}	=	44	lb/ft	60	lb/ft	
Moment Demand	=	4041	lb-ft	5436	lb-ft	
Shear Demand	=	862	lb	1160	lb	"Vs"
Loads on Caps						
w_{live}	=	1500	lb/ft	1500	lb/ft	
w_{dead}	=	687	lb/ft	707	lb/ft	
Moment Demand	=	22143	lb-ft	22346	lb-ft	
Shear Demand	=	11810	lb	11918	lb	

CASE 2A - POINT LOADING (TRUCK PARALLEL TO STRINGERS)

		4x12 Stringers		6x12 Stringers	
Loads on Stringers					
Moment Demand	=	18572	lb-ft	28717	lb-ft
Shear Demand	=	4601	lb	7117	lb
Loads on Caps					
Moment Demand	=	39845	lb-ft	44848	lb-ft
Shear Demand	=	22960	lb	26030	lb

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SUMMARY - 4x12 Stringers

	Shear			Moment			Deflection	
	Demand	Capacity*	DCR	Demand	Capacity*	DCR	Demand	Ratio L/...
	lb	lb	-	lb-ft	lb-ft	-	in	-
Case 1 - Distributed Loads								
Decking	80	4190	0.02	17	2652	0.01		
Stringer	862	5587	0.15	4041	11613	0.35	0.19	923
Cap	11810	16320	0.72	22143	38400	0.58		
Case 2A - Point Loads (Truck Axle parallel to Cap)								
Decking **	5976	20114	0.30	9011	12730	0.71		
Stringer	4601	8940	0.51	18572	18580	1.00	0.60	298
Cap	22960	26112	0.88	39845	61440	0.65		

*Increase capacity by factor of 1.6 for Case 2 (truck loads). This is due to an increase in the load duration factor, C_D, from 1 to 1.6, based on NDS 2.3.2, considering a ten minute load duration

Adjusted C_D for Case 2: 1.6

** Values for Demand are from SAP analysis. Capacities are multiplied by 3 deck boards

SUMMARY - 6x12 Stringers

	Shear			Moment			Deflection	
	Demand	Capacity*	DCR	Demand	Capacity*	DCR	Demand	Ratio L/...
	lb	lb	-	lb-ft	lb-ft	-	in	-
Case 1 - Distributed Loads								
Decking	106	4190	0.03	30	2652	0.01		
Stringer	1160	8160	0.14	5436	19200	0.28	0.21	863
Cap	11918	16320	0.73	22346	38400	0.58		
Case 2A - Point Loads (Truck Axle parallel to Cap)								
Decking **	8014	20114	0.40	9796	12730	0.77		
Stringer	7117	13056	0.55	28717	30720	0.93	0.73	246
Cap	26030	26112	1.00	44848	61440	0.73		

*Increase capacity by factor of 1.6 for Case 2 (truck loads). This is due to an increase in the load duration factor, C_D, from 1 to 1.6, based on NDS 2.3.2, considering a ten minute load duration

Adjusted C_D for Case 2: 1.6

** Values for Demand are from SAP analysis. Capacities are multiplied by 3 deck boards

2.1.4.2 CASE 2B Governing

Key
- 🟧 user input
- 🟩 user input to adjust
- ⬜ calculated formula

Design Parameters - General

Sb	Bent Spacing	=	15	ft	
Sp	Pile Spacing	=	9	ft	
ww	wheel width	=	1.667	ft	AASHTO 2007, 3.6.1.2.5
aw	axle width	=	6	ft	

Design Parameters - 4x12 Stringers

P	Truck Axle/2	=	14500	lb	Fire Truck	
Ss	Stringer Spacing	=	1	ft		
a2	dist. to axle for stringers (Case 2B)	=	6	ft	a>b, yes	Adjust these values to find max demand
a3	distance to axle for cap (Case 2A)	=	2	ft	a>b, yes	
$n_{Case\ 2A}$	% of P in 1 stringer	=	0.25		based on Caltrans 2004 Bridge Design Specifications, Table 3.23.1 and SAP analysis	
$n_{Case\ 2B}$	% of P in 1 stringer	=	0.25			

Design Parameters - 6x12 Stringers

P	Truck Axle/2	=	17800	lb	Fire Truck	
Ss	Stringer Spacing	=	1.333	ft		
a2	dist. to axle for stringers (Case 2B)	=	6.4	ft	a>b, yes	Adjust these values to find max demand
a3	distance to axle for cap (Case 2A)	=	2	ft	a>b, yes	
$n_{Case\ 2A}$	% of P in 1 stringer	=	0.34		based on Caltrans 2004 Bridge Design Specifications, Table 3.23.1 and SAP analysis	
$n_{Case\ 2B}$	% of P in 1 stringer	=	0.34			

Wdougfir	Unit wt of doug fir	=	32	pcf	
Wac	unit weight of AC	=	150	pcf	
Live	live load	=	100	psf	*CBC 2010 Table 1607.1*

Bent Cap Properties		12x12			
hc	height	=	12	in	
wc	width	=	12	in	

Stringer Properties		4x12			
hs	height	=	12	in	
ws	width	=	4	in	
E	Modulus of Elasticity	=	1900000	psi	NDS Table 4A, Douglas Fir Larch (North)
I	Moment of Inertia	=	576	in^4	$ws*hs^3/12$

Stringer Properties		6x12			
hs	height	=	12	in	
ws	width	=	6	in	
E	Modulus of Elasticity	=	1600000	psi	NDS Table 4D, Douglas Fir Larch (North)
I	Moment of Inertia	=	864	in^4	$ws*hs^3/12$

Decking Properties		3x12		
hd	height	=	3	in
wd	width	=	12	in

Deck Topping Properties		AC		
ht	height	=	2	in

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CASE 1 - DISTRIBUTED LOADING

		4x12 Stringers		6x12 Stringers		
Loads on Decking						
w_{live}	=	100	lb/ft	100	lb/ft	
w_{dead}	=	33	lb/ft	33	lb/ft	
Moment Demand	=	17	lb-ft	30	lb-ft	
Shear Demand	=	80	lb	106	lb	
Loads on Stringers						
w_{live}	=	100	lb/ft	133	lb/ft	
w_{dead}	=	44	lb/ft	60	lb/ft	
Moment Demand	=	4041	lb-ft	5436	lb-ft	
Shear Demand	=	862	lb	1160	lb	"Vs"
Loads on Caps						
w_{live}	=	1500	lb/ft	1500	lb/ft	
w_{dead}	=	687	lb/ft	707	lb/ft	
Moment Demand	=	22143	lb-ft	22346	lb-ft	
Shear Demand	=	11810	lb	11918	lb	

CASE 2A - POINT LOADING (TRUCK PARALLEL TO STRINGERS)

		4x12 Stringers		6x12 Stringers	
Loads on Stringers					
Moment Demand	=	14822	lb-ft	24382	lb-ft
Shear Demand	=	3667	lb	6038	lb
Loads on Caps					
Moment Demand	=	32734	lb-ft	38803	lb-ft
Shear Demand	=	18516	lb	22252	lb

CASE 2B - POINT LOADING (TRUCK PERPENDICULAR TO STRINGERS)

		4x12 Stringers		6x12 Stringers		
Loads on Stringers						
Moment Demand	=	18628	lb-ft	30608	lb-ft	
Shear Demand	=	5337	lb	8833	lb	"Vmax"

moffatt & nichol

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SUMMARY - 4x12 Stringers

	Shear			Moment			Deflection	
	Demand	Capacity*	DCR	Demand	Capacity*	DCR	Demand	Ratio L/...
	lb	lb	-	lb-ft	lb-ft	-	in	-
Case 1 - Distributed Loads								
Decking	80	4190	0.02	17	2652	0.01		
Stringer	862	5587	0.15	4041	11613	0.35	0.19	923
Cap	11810	16320	0.72	22143	38400	0.58		
Case 2A - Point Loads (Truck Axle parallel to Cap)								
Decking **	4684	20114	0.23	7063	12730	0.55		
Stringer	3667	8940	0.41	14822	18580	0.80	0.49	365
Cap	18516	26112	0.71	32734	61440	0.53		
Case 2B - Point Loads (Truck Axle parallel to Stringers)								
Decking **	5945	20114	0.30	7975	12730	0.63		
Stringer	5337	8940	0.60	18628	18580	1.00	0.73	247
Cap	Case 2A governs over 2B for cap							

*Increase capacity by factor of 1.6 for Case 2 (truck loads). This is due to an increase in the load duration factor, C_D, from 1 to 1.6, based on NDS 2.3.2, considering a ten minute load duration

Adjusted C_D for Case 2: 1.6

** Values for Demand are from SAP analysis. Capacities are multiplied by 3 deck boards

SUMMARY - 6x12 Stringers

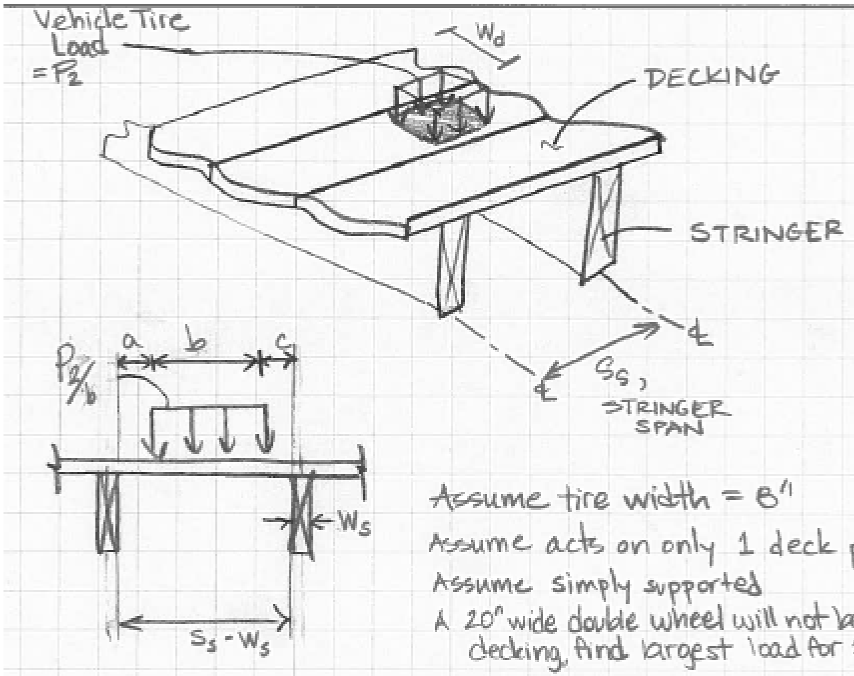
	Shear			Moment			Deflection	
	Demand	Capacity*	DCR	Demand	Capacity*	DCR	Demand	Ratio L/...
	lb	lb	-	lb-ft	lb-ft	-	in	-
Case 1 - Distributed Loads								
Decking	106	4190	0.03	30	2652	0.01		
Stringer	1160	8160	0.14	5436	19200	0.28	0.21	863
Cap	11918	16320	0.73	22346	38400	0.58		
Case 2A - Point Loads (Truck Axle parallel to Cap)								
Decking **	6728	20114	0.33	8225	12730	0.65		
Stringer	6038	13056	0.46	24382	30720	0.79	0.63	285
Cap	22252	26112	0.85	38803	61440	0.63		
Case 2B - Point Loads (Truck Axle parallel to Stringers)								
Decking **	7298	20114	0.36	9790	12730	0.77		
Stringer	8833	13056	0.68	30608	30720	1.00	0.94	191
Cap	Case 2A governs over 2B for cap							

*Increase capacity by factor of 1.6 for Case 2 (truck loads). This is due to an increase in the load duration factor, C_D, from 1 to 1.6, based on NDS 2.3.2, considering a ten minute load duration

Adjusted C_D for Case 2: 1.6

** Values for Demand are from SAP analysis. Capacities are multiplied by 3 deck boards

2.2 Check Decking for 1 Wheel (Smaller Vehicle)



Assume tire width = 8"

Assume acts on only 1 deck plank

Assume simply supported

A 20" wide double wheel will not be a point load on decking, find largest load for 1 wheel

Maximum Moment $= \frac{\frac{P_2}{b}\cdot b}{2\cdot(S_s-W_s)}(2\cdot c+b)\cdot\left[a+\frac{\frac{P/b\cdot b\cdot(2\cdot c+b)}{2(S_s-W_s)}}{2\cdot P/b}\right]$

$= \frac{P_2}{2(S_s-W_s)}(2c+b)\left[a+\frac{1\cdot b(2c+b)}{4\cdot(S_s-W_s)}\right]$

Maximum Shear when $a<c$ $= \frac{P_2\cdot(2c+b)}{2\cdot(S_s-W_s)}$

→ This calculation is conservative for shear, as it should be @d from the face.

(Equations from AISC 360-05)

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DESIGN CHECK - Decking Tire Load

Key

user input

calculated formula

Design Parameters

a	dist to tire	=	0.010	ft
b	tire width	=	0.6667	ft
c	Ss -ws - a -b	=	0.32	ft
Ss	Stringer spacing	=	1.33	ft
ws	width of stringer	=	0.33	ft

a needs to be < c

Check: OK

DOUGLAS FIR (Existing)

P2 Passenger Vehicle 1 Tire Load	=	10212	lb	<-- Maximum Allowable Load from 1 Tire on 1 plank
Moment Capacity	=	4243	lb-ft	*multiplied by 1.6 load duration factor*
Shear Capacity	=	6705	lb	*multiplied by 1.6 load duration factor*
Moment Demand	=	1534	lb-ft	
Shear Demand	=	6705	lb	
DCR Moment	=	0.36		
DCR Shear	=	1.00		Shear governs, Goal seek to find P2

IPE

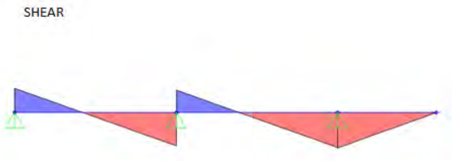
Ipe nominal bending stress = 3700 psi

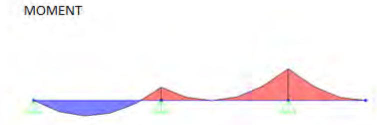
Doug Fir nominal bending stress = 2000 psi

Ipe is stronger than Doug Fir in bending, therefore the allowable tire load will be greater than : 10212 lb

Note that this is a truck with only 2 tires per rear axle and is conservative as most likely the tires are wider than 8 inches for a heavy truck, and the load will distribute through the asphalt. It will most likely distribute to at least 2 deck boards. This calculation is for a single deck board.

2.3 Cantilever Cap – (No Truck Loads)


SHEAR


MOMENT

Assumptions:

Sp = 9 ft

SINGLE CAP, NO BRACE

Loads

w live = 1500 lb/ft

w dead = 605 lb/ft

SAP Analysis Results:

Cantilever length	Moment Demand	Moment Capacity	DCR	Shear Demand	Shear Capacity	DCR
ft	lb-ft	lb-ft	-	lb	lb	-
2	4278	33797	0.1	4278	14988	0.3
4	17112	33797	0.5	8556	14988	0.6
5.5	32360	33797	1.0	11770	14988	0.8
6	38502	33797	1.1	12834	14988	0.9
8	68448	33797	2.0	17112	14988	1.1

Max cantilever length for single cap is 5.5' from CL of pile

I checked if the other pile spacing affected it, and there was no difference between 7' and 9' spacing

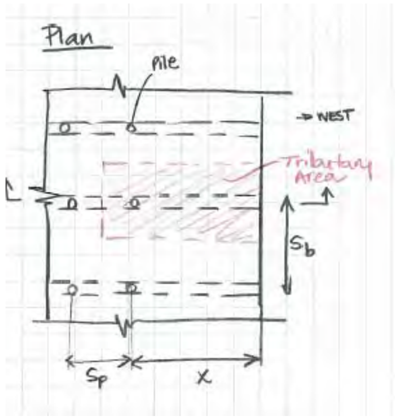
---

DOUBLE CAP, NO BRACE

w live = 1500 lb/ft

w dead = 634 lb/ft

SAP Analysis Results, just adjust by added deadweight



Cantilever length	Moment Demand	Moment Capacity	DCR	Shear Demand	Shear Capacity	DCR
ft	lb-ft	lb-ft	-	lb	lb	-
2	4486	67594	0.1	4486	29977	0.1
4	17943	67594	0.3	8972	29977	0.3
5.5	33932	67594	0.5	12342	29977	0.4
6	40372	67594	0.6	13457	29977	0.4
7.5	63082	67594	0.93	16824	29977	0.6
8	71773	67594	1.1	17943	29977	0.6

Adjustment Factor = (Wdead new /Wdead old)

= 1.05

Max cantilever length for double cap is 7.5' from CL of pile

Capacities are doubled with a double cap

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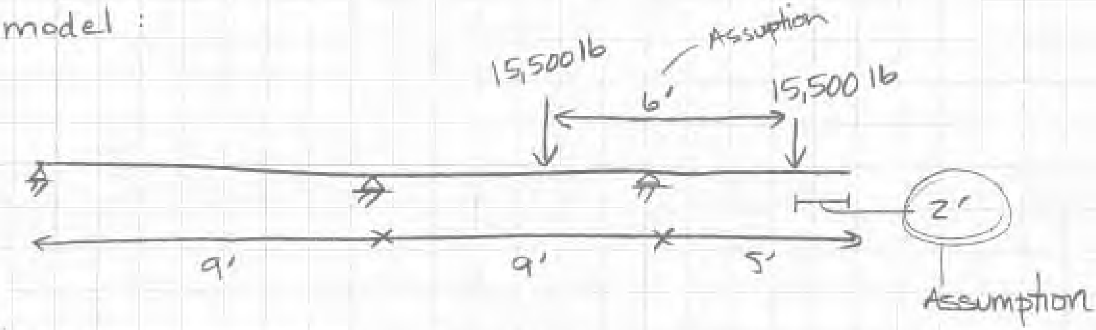
For the east side new Promenade, check if a 5’ cantilever is adequate for fire truck load (31,000 lb rear axle)

Check cap cantilever for truck :

SAP model :



<u>Demand</u>

Max M = 54488 lb-ft

Max V = 18700 lb

<u>Capacity</u>

M_{allow} = 61,440 lb-ft

V_{allow} = 26,080 lb

<u>DcR</u>

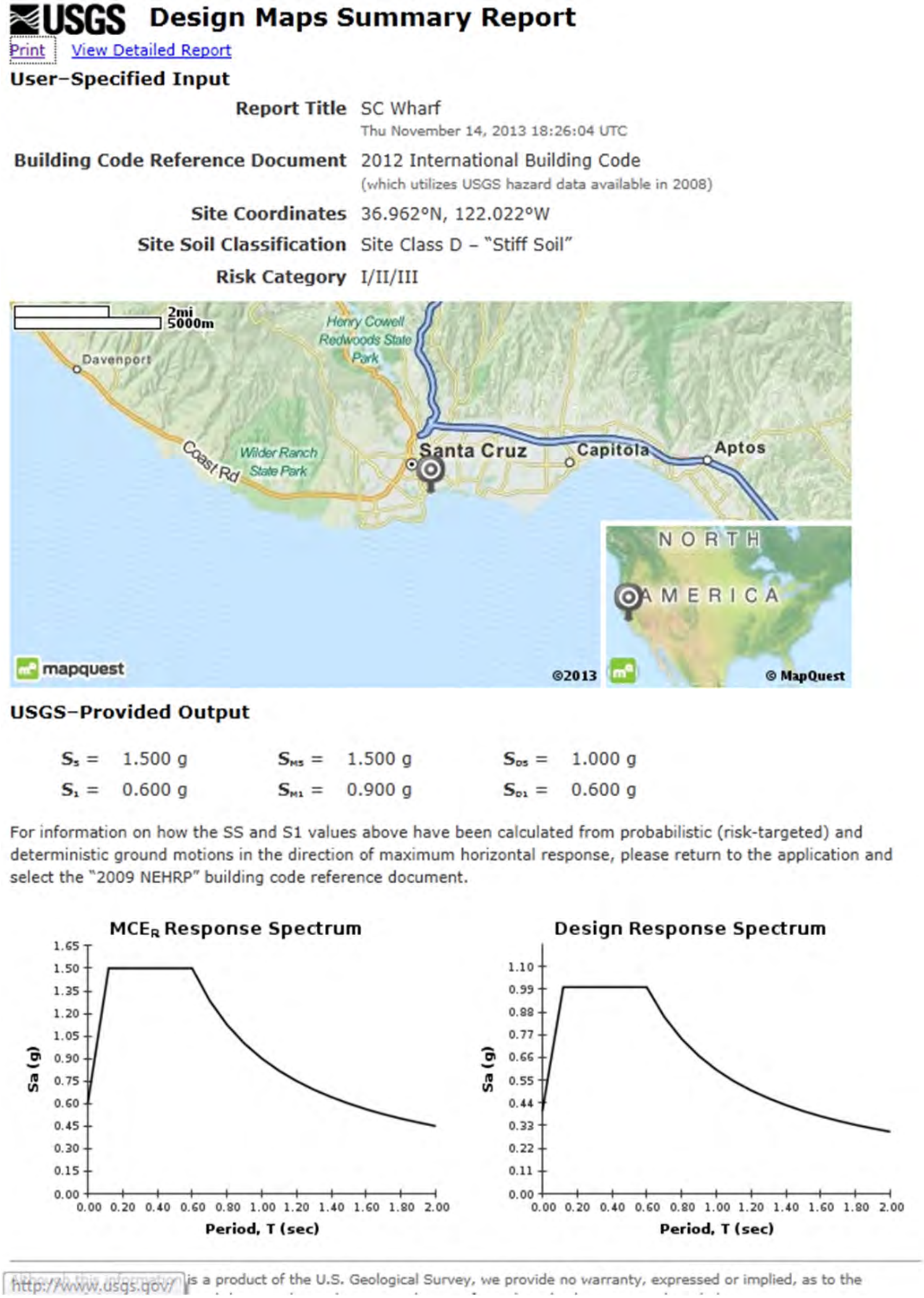
Moment = 54488/ 61,440 = 0.89 , ok

Shear = 18700/ 26,080 = 0.72 ok

Cantilever at Promenade is okay if truck axle spacing is 15’ or greater.

3 Single Pile Seismic Analysis

3.4 Seismic Spectra



USGS Design Maps Summary Report

Print View Detailed Report

User–Specified Input

Report Title	SC Wharf Thu November 14, 2013 18:26:04 UTC
Building Code Reference Document	2012 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	36.962°N, 122.022°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

$S_S =$ 1.500 g $S_{MS} =$ 1.500 g $S_{DS} =$ 1.000 g

$S_1 =$ 0.600 g $S_{M1} =$ 0.900 g $S_{D1} =$ 0.600 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

http://www.usgs.gov/ is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the

3.5 Effective Vertical Loads on 1 Pile

Effective Seismic Weight, W (on 1 Pile)

Substructure Dead:

w = 687 lb/ft along a cap
Cap Tributary Span = 10 ft

weight = 6870 lb per pile

Building Dead:
1-Story
Interior Column Load = 6500 lb *from Mesiti-Miller calcs*
OR
Bearing Wall Load = 400 lb/ft "
Wall Trib. Length = 15 ft (say wall runs N-S)
= 6000 lb

Governing Bldg Load = 6500 lb per pile

2-Story
Interior Column Load = 18700 lb *from Mesiti-Miller calcs*
OR
Bearing Wall Load = 800 lb/ft "
Wall Trib. Length = 15 ft (say wall runs N-S)
= 12000 lb

Governing Bldg Load = 18700 lb per pile

1/3 of Pile Weight to Pt of Fixity
Length to 5D = 58 ft
Unit Weight = 32 pcf
X-Section Area = 113 in^2

Weight *1/3 = 486 lb

Building Live:

Interior Column Load = 6400 lb *from Mesiti-Miller calcs*

OR

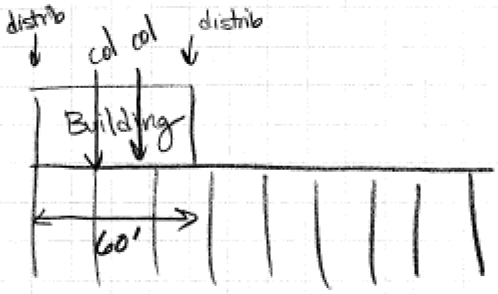
Bearing Wall Load = 200 lb/ft "

Wall Trib. Length = 15 ft (say wall runs N-S)

= 3000 lb

Governing Bldg Live Load = 6400 lb per pile

DISTRIBUTE BENT DEAD LOAD ONTO SINGLE PILE (for seismic analysis)



- Bent spacing = 15'
- Building Loads from Mesiti-Miller calcs (11-13-13)

1 story Bldg weight $= 15\text{ft}\cdot\underbrace{(400\text{ lb/ft})\cdot 2}_{\text{distrib wall}} + \underbrace{(6{,}500\text{ lb})\cdot 2}_{\text{interior col.}} = \underline{25\text{ Kips}}$ per bent

Say Bent @ Building area has 20 piles (Bent 88)

Pile Head Mass = 6.9 k + 0.5 k + 25 k / 20 piles

(substruc. dead) + 1/3 pile self wt. + building

= **8.7 k/pile for 1-story bldg**

2 story building weight = 15 ft * (800 lb/ft)* 2 + (18,700 lb)*2 = 61.4 kips/bent

Pile Head Mass = 6.9 k + 0.5 k + 61.4 k / 20 piles

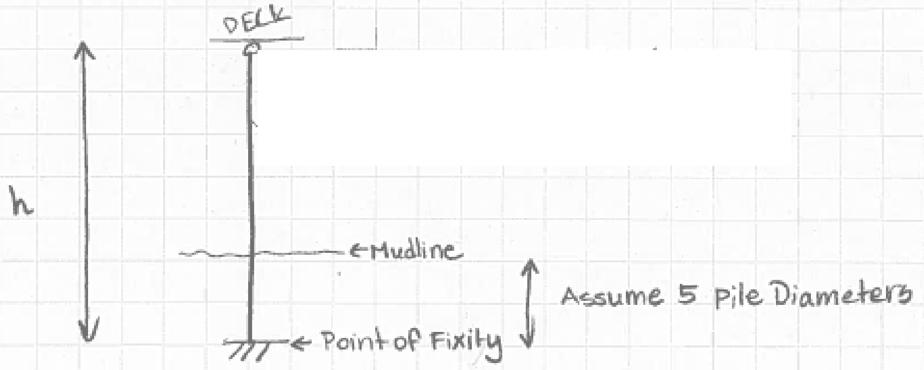
= **10.5 k/pile for 2-story bldg**

3.6 SAP Model Properties

CREATE MODEL OF A SINGLE WHARF PILE. FIND PUSHOVER CAPACITY

FIND PUSHOVER CAPACITY

DETERMINE EFFECTIVE MASS FOR ONE PILE AND APPLY EQ TO GET DISPL. DEMAND.



$E = 1600000$ psi (NDS 2005, Table 4D, Douglas Fir Larch (North)]

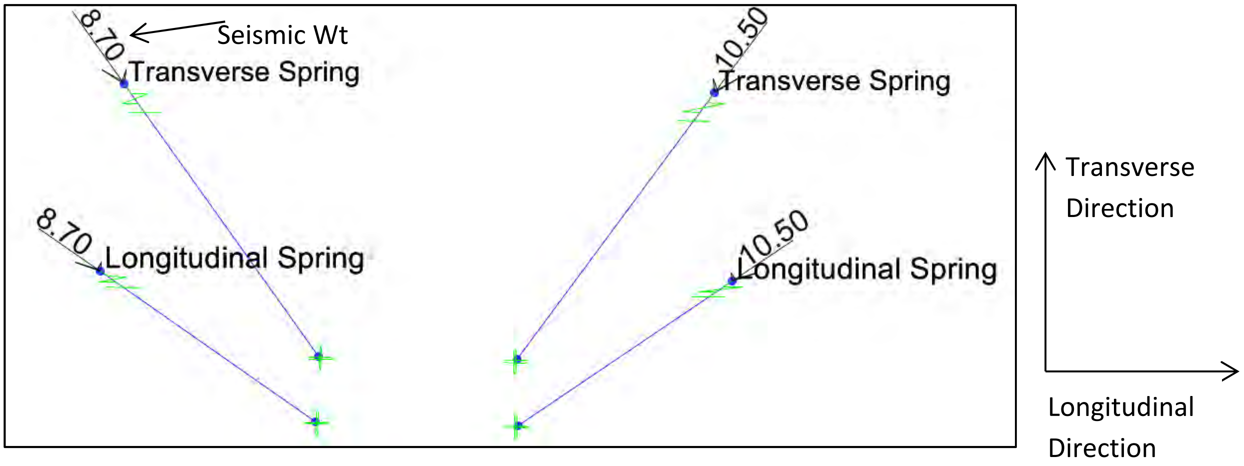
$I = \pi/4 \cdot R^4 = \pi/4 \, (12/2)^4 = 1018 \text{ in}^4$

$EI = 11310$ Kip·ft²

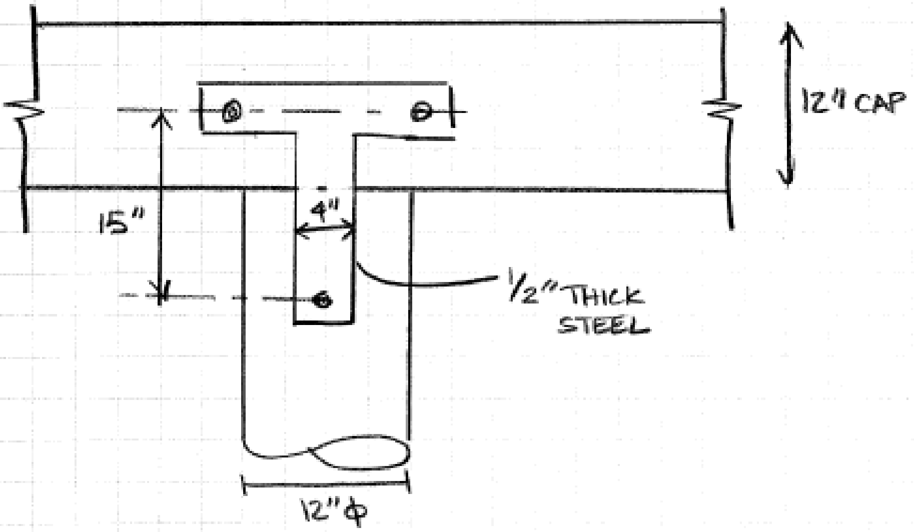
$h = 5 \cdot \text{Diam} + \text{Depth to ML}$

$5 \cdot 1$ ft ↳ 23' + 30'

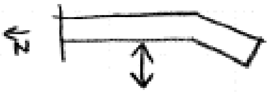
$= 58$ ft

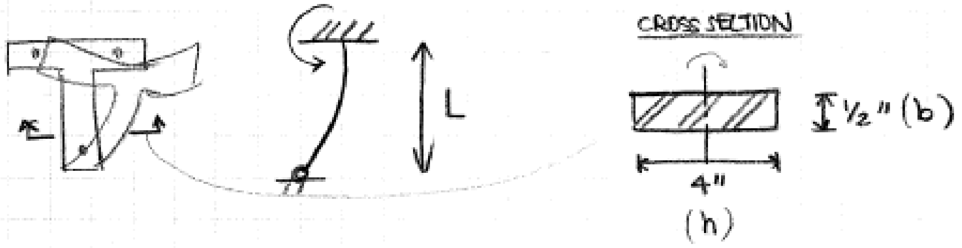


ESTIMATE EFFECTIVE STIFFNESS AT PILE T-CONNECTION



EARTHQUAKE IN TRANSVERSE DIRECTION





$K = \frac{EI}{5L} \cdot 2$ Tee's

Factor of 5 reduction is based on engineering judgment

$L = 15$ in

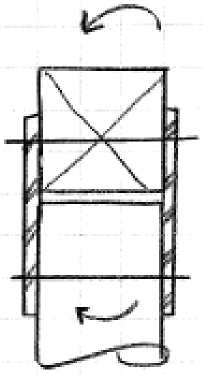
$E = 29{,}000$ ksi

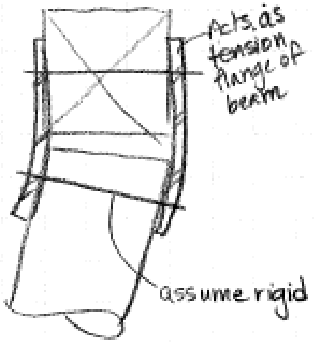
$I = bh^3/12 = (0.5\text{in})(4.\text{in})^3/12 = 2.67 \text{ in}^4$

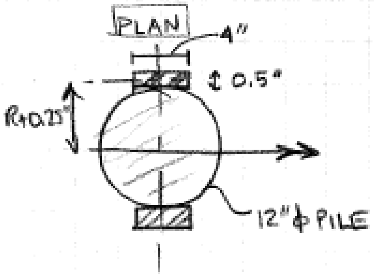
$K = (29000 \text{ ksi}) \cdot (2.67 \text{in}^4) / [5 \cdot 15 \text{ in})] \cdot 2$ Tee's $\boxed{= 2064 \text{ kip}\cdot\text{in/rad}}$

ESTIMATE EFFECTIVE STIFFNESS AT PILE T-CONNECTION

EARTHQUAKE IN LONGITUDINAL DIRECTION







$K = \frac{EI}{20L}$

Factor of 20 reduction is based on engineering judgment

$L = 15\ in$

$E = 29000\ Ksi$

$I = 2(A \cdot [r + 0.25 in]) = 2[(4 \cdot 0.5)(\frac{12}{2} + 0.25)]^2 = 156\ in^4$

$K = \frac{EI}{20L} = \frac{\overset{ksi}{29000} \cdot 156\ in^4}{20 \cdot 15\ in} = 15{,}080\ Kip \cdot in / radian$

3.7 Demand vs. Capacity – Single Pile Seismic

1 Story Building

Demand	unit	Transverse (Y direction)	Longitudinal (X direction)	SRSS (Combine 100% + 30%)	
				30X + 100Y	**100X + 30Y**
Displacement	**in**	38	29	**39**	**31**
Moment	**lb-ft**	24056	24889	**25188**	**25914**
Shear	**lb**	553	760	**598**	**778**

2 Story Building

Demand	unit	Transverse (Y direction)	Longitudinal (X direction)	SRSS (Combine 100% + 30%)	
				30X + 100Y	**100X + 30Y**
Displacement	**in**	43	32	**44**	**34**
Moment	**lb-ft**	26667	27333	**27899**	**28480**
Shear	**lb**	607	867	**660**	**886**

Shear and Moment Demands are divided by (R/I):

R = 1.5 (ASCE 7-10 Table 12.1-1 timber cantilever structure)

I = 1 Importance Factor

Demands - Seismic 1 - Story Building

Pu, Dead = 8700 lb

Pu, Live = 6400 lb (from Mesiti-Miller calcs)

EQ, from SAP Response Spectra

Vu = 778 lb from SAP (SRSS, 100%+30%)

Mu = 25914 lb-ft "

Load Combinations (ASD) IBC 2012 (should be same as CBC 2013)

		Pu	Mu	Vu
		lb	lb-ft	lb
1)	D + 0.7E	8700	18140	545
2)	D + 0.75*0.7*E+0.75L	13500	13605	408
3)	0.6D + 0.7E	5220	18140	545

Check below to determine which governs for the pile:

Case 1 governs

Pu = 8700 lb

Mu = 18140 lb-ft

A = 113 in^2 *Area*

S = 170 in^3 *Section Modulus*

fc = 77 psi *Pu/A*

fb = 1283 psi *Mu/S*

Capacity increase by 1.6 for Cd, load duration factor for EQ (NDS 2.3.2)

Pn = 65676 lb

Mn = 42672 lb-ft

Vn = 9755 lb

Per NDS 3.9.2, members with compression and bending must have (one directional moment):

$$\left[\frac{f_c}{F'_c}\right]^2 + \frac{f_b}{F'_b[1-(f_c/F_{cE})]} \le 1.0$$

Check:

DCR: 0.50 < 1 **OK**

Fc' = 581 psi incr. by 1.6 Cd

Emin = 790000 psi

Fce = 630 psi incr. by 1.6 Cd

Le = 40.6 ft L*Ke *NDS 3.7.1*

d=b = 12 in

Fb' = 3018 psi incr. by 1.6 Cd

Demands - Seismic - 2 Story Building

Pu, Dead = 10500 lb

Pu, Live = 6400 lb (from Mesiti-Miller calcs)

EQ, from SAP Response Spectra

Vu = 886 lb from SAP (SRSS, 100%+30%)

Mu = 28480 lb-ft "

Load Combinations (ASD) IBC 2012 (should be same as CBC 2013)

		Pu	Mu	Vu
		lb	lb-ft	lb
1)	D + 0.7E	10500	19936	620
2)	D + 0.75*0.7*E+0.75L	15300	14952	465
3)	0.6D + 0.7E	6300	19936	620

Check below to determine which governs for the pile:

Case 1 governs

Pu = 10500 lb

Mu = 19936 lb-ft

A = 113 in^2 *Area*

S = 170 in^3 *Section Modulus*

fc = 93 psi *Pu/A*

fb = 1410 psi *Mu/S*

Capacity increase by 1.6 for Cd, load duration factor for EQ (NDS 2.3.2)

Pn = 65676 lb

Mn = 42672 lb-ft

Vn = 9755 lb

Per NDS 3.9.2, members with compression and bending must have (one directional moment):

$$\left[\frac{f_c}{F'_c}\right]^2 + \frac{f_b}{F'_b[1-(f_c/F_{cE})]} \leq 1.0$$

Check:

DCR: **0.57 < 1** **OK**

Fc' = 581 psi incr. by 1.6 Cd

Emin = 790000 psi

Fce = 630 psi incr. by 1.6 Cd

Le = 40.6 ft L*Ke *NDS 3.7.1*

d=b = 12 in

Fb' = 3018 psi incr. by 1.6 Cd

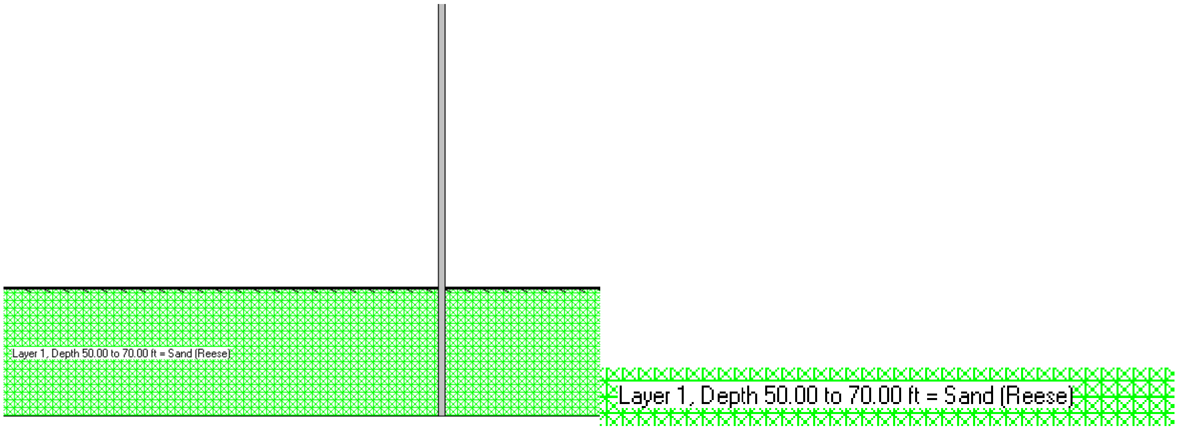
3.8 LPile Check

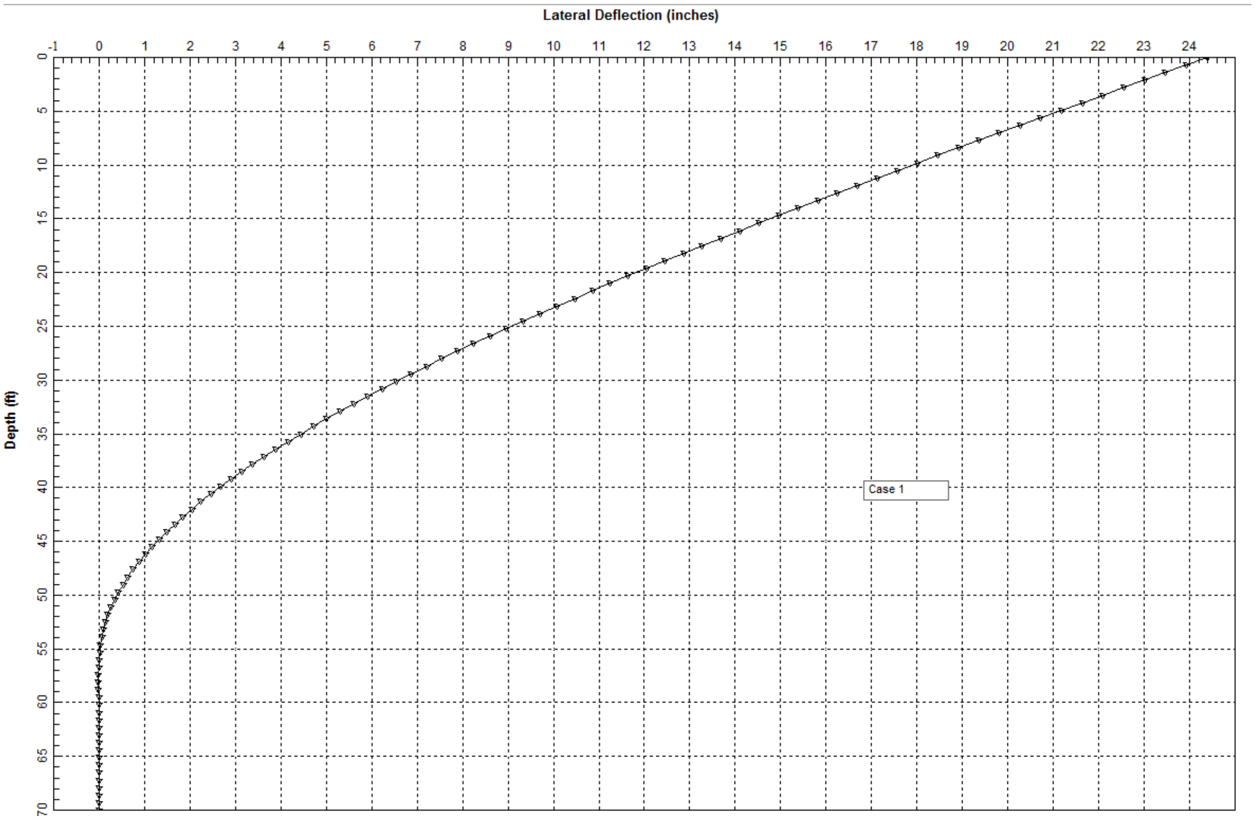
Purpose is to confirm that fixity is at 5 pile diameters below the soil

Assume soil sand properties

Free head

Apply 400 lb load



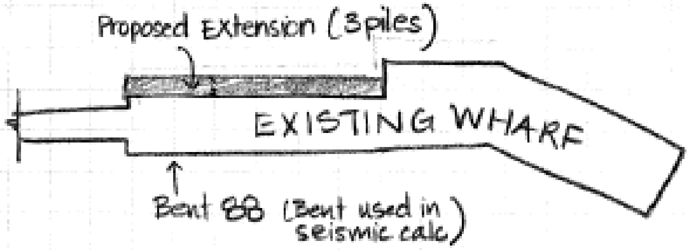


FIXITY AT 5 DIAMETERS IS CONFIRMED - soil starts at depth 50’, deflection is 0” by depth of 55’

3.9 Master Plan's Effect on Seismic Displacement

As part of the Master Plan (at this stage), it is proposed to add an extension to the east side of the wharf that is 3 piles wide. The following calculation estimates the added stiffness this will add to the wharf, regarding the displacements during an earthquake.

Calculate Effect of Adding 3 piles to Each Wharf Bent, as part of the Wharf Master Plan:



$$\text{Pile Head Mass} = \underbrace{6.9^k}_{\text{substruc. dead}} + \underbrace{0.5^k}_{1/3\text{ pile self-wt.}} + \underbrace{25^k}_{\text{building}} / \overset{\text{previously was 20}}{\boxed{23 \text{ piles}}}$$

$= 8.5$ k/pile for single story building ←

(vs 8.7 k/pile existing wharf)

Run value in SAP to compare with existing structure:

Results

	EXISTING WHARF DISPL.	RETROFIT Wharf Displ.	% Reduction in Movement
Transverse Displ. Y	38.4"	38.0"	1%
Longitudinal Displ. X	29.0"	28.7"	1%

approximately 1% reduction in displacement

3.10 Conclusions

This analysis is a simplified analysis and should not be used for design or to replace a comprehensive seismic analysis.

- Assuming partial-fixity connections at top, the single pile analysis results are:
  - One Story Building
    - Transverse Direction Displacement = 39 inches
    - Longitudinal Direction Displacement = 31 inches
    - Demands are less than the capacities for the piles.
  - Two Story Building
    - Transverse Direction Displacement = 44 inches
    - Longitudinal Direction Displacement = 34 inches
    - Demands are less than the capacities for the piles.
- Adding 3 additional piles to the east side of the wharf per the Master Plan reduces the seismic displacements by approximately 1%.

4 Building Load Analysis

Building loads (dead and live) were calculated by Mesiti-Miller (11-13-2013). Here, the building loads acting on the substructure are examined.

Key

user input
calculated formula

Design Parameters

Sb	Bent Spacing	=	15	ft
Sp	Pile Spacing	=	9	ft
Ss	Stringer Spacing	=	1.33	ft

			Depth of Member	
Cap Size	=	12x12	11.5	= hc (in.)
Stringer Size	=	4x12	11.5	= hs (in.)
		6x12	11.5	= hs (in.)

Demands

Distributed Dead Load for Cap Beam (substructure)	=	605	lb/ft length of cap	*see cantilever cap check*
Distributed Dead Load for Stringer (substructure)	=	51	lb/ft length of stringer	*see vertical load calcs*

Point Load for Interior Column (dead + live)

$P_{2\,story}$	=	50,100	lb
$P_{1\,story}$	=	12,900	lb

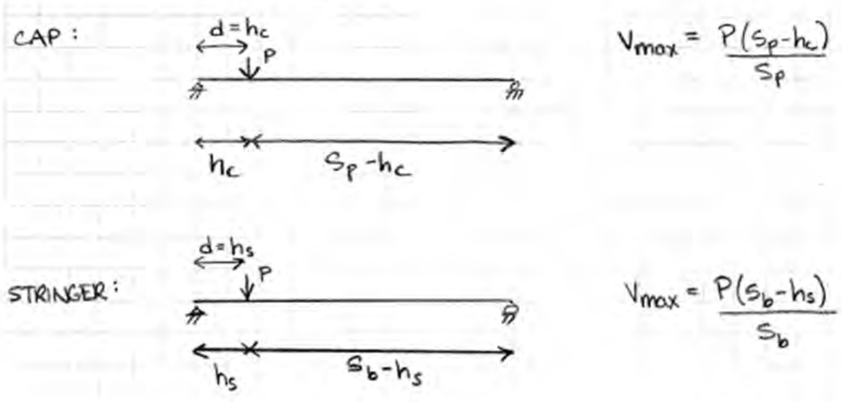
Distributed Load for Exterior Bearing Wall (dead + live)

$w_{2\,story}$	=	2000	lb/ft
$w_{1\,story}$	=	600	lb/ft

Equations

Point Load

Vu = P*(S-h)/S -->



Mu = PL/4, conservatively assume simply-supported

Distributed Load

Vu = 0.6*w*L, for continuous span

Mu = $wL^2/8$, conservatively assume simple span

Demands for Substructure Dead Loads

Cap Vu sub dead = 2724 lb
Mu sub dead = 6128 lb-ft
Stringer Vu sub dead = 383 lb, neglect weight difference btwn 4x12 and 6x12
Mu sub dead = 1436 lb-ft

Capacities

Cap Vn = 14988 lb
Mn = 33797 lb-ft
Stringer, 4x12 Vn = 4685 lb
Mn = 9332 lb-ft
Stringer, 6x12 Vn = 7168 lb
Mn = 16164 lb-ft

Available Capacity after Reduced for Substructure Dead Load Demand

Cap Vn - Vu sub dead = 12265 lb
Mn - Mu sub dead = 27669 lb-ft
Stringer, 4x12 Vn - Vu sub dead = 4302 lb
Mn - Mu sub dead = 7896 lb-ft
Stringer, 6x12 Vn - Vu sub dead = 6785 lb
Mn - Mu sub dead = 14728 lb-ft

Load Directly on:

Cap		Vu*	Vn**	DCR	Mu*	Mn**	DCR
		lb	lb	-	lb-ft	lb-ft	-
One-Story Building	Interior Column Load	11526	12265	0.94	29025	27669	1.05
	Bearing Wall Load	3240	12265	0.26	6075	27669	0.22
Two-Story Building	Interior Column Load	44765	12265	3.65	112725	27669	4.07
	Bearing Wall Load	10800	12265	0.88	20250	27669	0.73

* Building Loads only

** Reduced for Capacity *after* substructure dead load applied

Stringer 4x12		Vu*	Vn**	DCR	Mu*	Mn**	DCR
		lb	lb	-	lb-ft	lb-ft	-
One-Story Building	Interior Column Load	12076	4302	2.81	48375	7896	6.13
	Bearing Wall Load	5400	4302	1.26	16875	7896	2.14
Two-Story Building	Interior Column Load	46899	4302	10.90	187875	7896	23.79
	Bearing Wall Load	18000	4302	4.18	56250	7896	7.12

* Building Loads only

** Reduced for Capacity *after* substructure dead load applied

Stringer 6x12		Vu*	Vn**	DCR	Mu*	Mn**	DCR
		lb	lb	-	lb-ft	lb-ft	-
One-Story Building	Interior Column Load	12076	6785	1.78	48375	14728	3.28
	Bearing Wall Load	5400	6785	0.80	16875	14728	1.15
Two-Story Building	Interior Column Load	46899	6785	6.91	187875	14728	12.76
	Bearing Wall Load	18000	6785	2.65	56250	14728	3.82

* Building Loads only

** Reduced for Capacity *after* substructure dead load applied

CONCLUSIONS:

Bearing walls - can be situated anywhere on a cap up to a 10 ft pile span

Bearing walls - can be on stringers if:

1 story	4x12	3 bundled
	6x12	2 bundled
2 story	4x12	8 bundled
	6x12	4 bundled

Interior Column - cannot fall on stringer

Interior Column - can fall on 9' span cap, if

1 story	within 3.5' of pile centerline
2 story	within 6" of pile centerline

HOWEVER, Pile capacity is 20 tons soil, and about the same for buckling/axial - 2 story column load is too high for one existing pile

5 Berthing Energy

ASSUMPTIONS

Vessel is 100 LT, Coast Guard Marine Protector Class
Vessel length = 87 ft
Vessel Beam = 19.5 ft
Vessel Draft = 5.583 ft

Approach Velocity = 1 ft/sec

3 existing timber piles resist the impact load

Source:

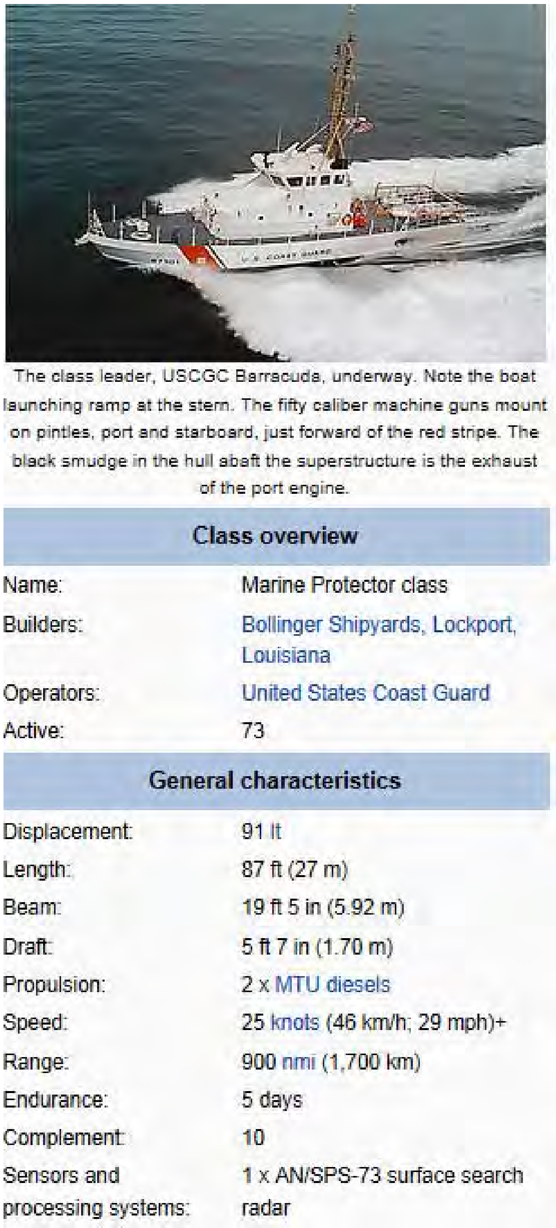
from Brad
wikipedia and coastguard website

CALCULATIONS

$E = 1/2 * W * V_n^2 / g$ Berthing Energy of Vessel

W	=	100	LT
Vn	=	1	ft/s
E	=	3.5	kip-ft

Vessel later checked for 200 LT,
DCRs were all below 1.0



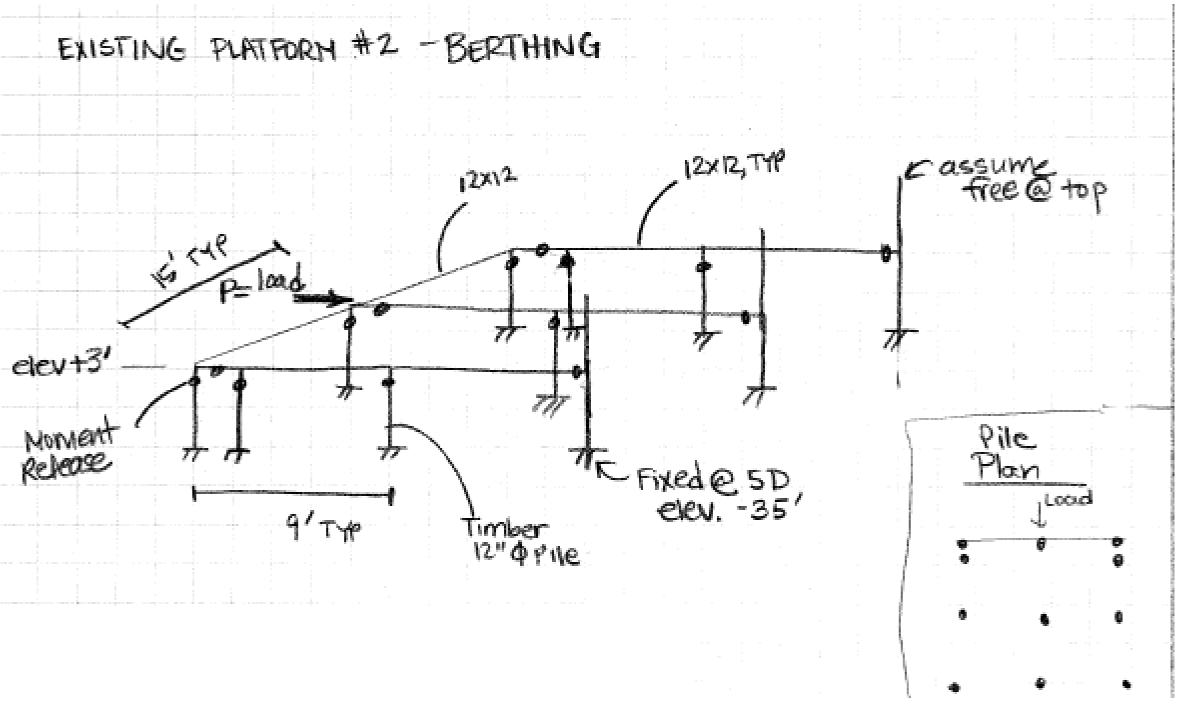
The class leader, USCGC Barracuda, underway. Note the boat launching ramp at the stern. The fifty caliber machine guns mount on pintles, port and starboard, just forward of the red stripe. The black smudge in the hull abaft the superstructure is the exhaust of the port engine.

Class overview	
Name:	Marine Protector class
Builders:	Bollinger Shipyards, Lockport, Louisiana
Operators:	United States Coast Guard
Active:	73

General characteristics	
Displacement:	91 lt
Length:	87 ft (27 m)
Beam:	19 ft 5 in (5.92 m)
Draft:	5 ft 7 in (1.70 m)
Propulsion:	2 x MTU diesels
Speed:	25 knots (46 km/h; 29 mph)+
Range:	900 nmi (1,700 km)
Endurance:	5 days
Complement:	10
Sensors and processing systems:	1 x AN/SPS-73 surface search radar

5.11 Berthing Analysis with Fenders and Existing Piles

- Create SAP model of existing Platform # 2 and determine the unit deflection and moment demands.
- Use these values to determine the wharf reaction for a berthing energy E (previous calculation)
- Compare demands and capacities of structural elements



Note: 12x12 waler does not exist currently. This is assumed to be installed or some equivalent.

SAP PROPERTIES

Existing Timber Pile

E	=	1500000	psi	
I		1018	in^4	pi/4 * R^4
EI	=	1.53E+09	lb - in^2	
diameter	=	12	in	
Mn	=	512	kip-in	

For SAP

I	=	1018	in^4	based on geometry 3.14*(do^4)/64)
E	=	1500000	psi	
Area	=	113	in^2	pi/4*$(D)^2$
Unit Wt	=	32	pcf	doug fir
S	=	170	in3	pi*d^3/32

Waler, 12x12 Timber

E	=	1500000	psi	
I		1457	in^4	
EI	=	2.19E+09	lb - in^2	
b=h	=	11.5	in	
Mn	=	649	kip-in	

For SAP

I	=	1457	in^4	based on geometry 3.14*(do^4)/64)
E	=	1500000	psi	
Area	=	132	in^2	pi/4*$(D)^2$
Unit Wt	=	32	pcf	doug fir
S	=	253	in3	$bh^2/6$

moffatt & nichol	CLIENT	Roma	JOB NO.	8181		
	PROJECT	Santa Cruz Wharf	DESIGNER	ETP	DATE	12/06/2013
			CHECKER	SS	DATE	1/2/2014

Analysis Table:

# Piles impacted	=	3

				Case 1 - High
E, Energy Demand	kip-ft	=	from berthing analysis	3.5
	kip-in	=		42

Pile Geometry

L	in	=		456
a	in	=		0
b	in	=		456

SAP Unit Load Analysis Results

Values from SAP for unit load

Deflection from SAP at impact pt	in	=	(for unit load)	2.11
Deflection from SAP at top	in	=	(for unit load)	2.11
Max M in Pile from SAP	kip-in	=	(for unit load)	46.20
Max M in Waler from SAP	kip-in	=	(for unit load)	63.00

Demands

	P	kip	=	iterate to get 0 error	3.6
	P corresponding to E	kip	=	2*E/(Deflection*# Piles Impacted) =	3.6
Goal seek ->	Error	kip	=	Corresponding P - P guess	0

Deflection at impact point	in	=	unit load deflection * P	7.7
Deflection at top	in	=	unit load deflection * P	7.7
Mu-pile, Actual M	kip-in	=	unit load M * P new	168
Mu -waler, Actual M	kip-in	=	unit load M * P new	229

Wharf Reaction	kip	=	$P*b^2/(2*L^3)*(a+2*L)$ * # piles	11

Demand/Capacity Check

Mn, pile	kip-in	=	from calculations	512
DCR for Moment-pile	**-**	**=**	**Actual M/Mn**	**0.33**
Mn, waler	kip-in	=	from calculations, mult by 1.6 Cd factor	649
DCR for Moment-waler	**-**	**=**	**Actual M/Mn**	**0.35**
Deflection at top	**in**	**=**	**Actual D**	**7.7**
Vn, Shear	kip	=	from calculations, mult by 1.6 Cd factor	13.76
DCR for Shear	**-**	**=**	**V(=P)/Vn**	**0.26**

Conclusions:

Vessel later checked for 200 LT, DCRs were all below 1.0

- The deflection would be ~ 8 inches
- Waler should be installed with some protective rub strips
- Moment and shear demand-to-capacity ratios for piles are under 1.0
- Existing structure with some upgrades is acceptable for 100 ton vessel

Attachment 2D Preliminary Evaluation, Vehicle Loading Memoranda

MEMORANDUM

To: **File**

From: **Brad Porter, P.E.; Erica Petersen, P.E.**

Date: **September 11, 2013**

Subject: **Santa Cruz Wharf – Preliminary Engineering Evaluation--DRAFT**

M&N Job No.: **8181**

Summary

Moffatt & Nichol (M&N) performed a preliminary investigation of the Santa Cruz Wharf ("the Wharf") on August 22, 2013 (from a boat) and Sept 5, 2013 (pavement) in preparation for the structural evaluations (above water and dive) that are scheduled for the fourth quarter of this year. The purpose of these investigations is to evaluate the overall structural condition of the Wharf and provide input to the master plan effort currently in progress. This memorandum outlines general observations made during these site visits and identifies items that will be more carefully observed during the inspection. An evaluation of the landings will be performed on September 18 and added to this memorandum at that time.

The following summary observations were made:

Wharf Structure

Piles-Most of the piles are in a serviceable condition, due to ongoing maintenance of the wharf. Occasional piles were observed that will require replacement but it does not appear that a large percentage will require replacement. An exception to this is below some of the buildings, particularly the Miramar Restaurant where driving piles below the building cannot be accomplished. In this area, there are missing piles that may compromise the structural integrity of the wharf.

Substructure (Caps and Stringers)-The caps and stringers that support the decking are in serviceable condition. There is evidence in locations, particularly below the buildings and kiosks, where wetting has occurred due to water leakage over time that has deteriorated the structural strength of these members. The tops of some caps appear soft and will require replacement; in some locations splices between the cap joints are unsupported below which creates discontinuity in the pile bent and reduces the structural capacity. These were observed in localized areas. The arrangement of the caps and the connections is varied, particularly along joints of different construction periods of the wharf. In an area below the east parking lot (near bent 120), stringers were observed that had been split due to what appears to have been an overstress, perhaps from a large point load (possibly from a truck).

Pavement

The asphalt pavement of the wharf roads and parking areas is severely cracked and deteriorated over the majority of the traffic areas. The amount of pavement that is serviceable is estimated to be less than 20%. Considering this, resurfacing of the entire pavement area is needed. This is mainly due to the flexible substructure (timber framing between piles) that supports the pavement, and the regular travel of trucks (garbage and delivery) along the road.

1. Wharf Investigation-Underside

The two engineers from M&N, accompanied by wharf staff (Britt H.), viewed the underside of the wharf from a boat in the morning during the lower tide (0.0 -3.0 MLLW). Various observations regarding the overall condition of the piles, caps, stringers, and piping were made. Photo 1 (all photos are located at the end of the memo) shows a typical bent with piles of different vintages. The Wharf was constructed in 1914 with various additions since then; pile age varies from 2 – 99 years. Old piles often still have creosote, while newer piles are identifiable by the greenish blue color indicating ACZA treatment. The current replacement piles at the wharf are timber coated in a black spray-on polyurea coating. In some locations where a pile is missing or had to be removed beneath a building, an “A-frame” was installed to distribute the load to the surrounding piles (Photo 2). After the boat inspection, M&N walked the topside of the wharf to observe the condition of the asphalt paving, layout of the topside structures, etc. M&N also made some observations below the wharf deck from some of the hatches (Photo 3) that are accessible on the topside. After the inspections, M&N met with Jon Bombaci, the wharf supervisor, to discuss the logistics of the inspections and his expectations for the project.

2. Engineering Evaluation Planning

After the under wharf investigation, M&N met with Jon Bombaci, the Wharf Supervisor, to discuss the logistics and procedures of the full engineering evaluations that will be conducted and his expectations for the project.

Inspection Logistics (Piles)

Discussed the intended inspection levels:

- Level I – 100% of piles, visual inspection from mudline to pile cap
- Level II – 10-20% of piles, to be determined as investigation develops, marine growth is removed from three bands and the condition of the underlying pile inspected and photographed
- Level III:
  - Coring – do 10 piles per time period (i.e. 10-1914 piles, 10-1970’s piles) to extract 2 inch diameter cores
  - Drill – remainder of 5% of piles (200)

Items to consider Below Deck:

- Caps
    - Connections – especially between different wharf construction time periods. Photo 4 shows a cap splice that is not preferable, as the splice is located mid-span between piles.
    - Rot, especially in areas of water leakage
    - Caps at end of wharf that were re-stitched together after the 1980's storms
- Stringers that are splitting or rotting (Photo 5)
- Pile deterioration (Photo 6 and Photo 7 show examples of this). At many deteriorated piles, a new pile has been installed right next to it.
- Check electrical below water grounds
- Pay close attention to structure under Miramar building.

Items to consider Above Deck:

- Pavement condition. The asphalt across the entire wharf is currently in poor condition, see Photo 8.
- Possibility of installing wheel runners along road (a 4x12 or similar) running in direction of wheels under wheels under pavement to alleviate asphalt cracking
- Feasible plans for repaving the entire wharf
    - Divide into sections
    - It might be possibly to chip out all AC and leave the wood decking exposed for a few months
- Wood decking or pavers to replace AC are not desired
- Sea-level rise – is there a need to raise the deck in some way
- Drainage of wharf deck
    - Currently there is no drainage system – it runs off into the ocean
    - Be prepared for what the agencies will require, but do not assume an extensive drainage system will be required
    - Different options for sloping the top surface
- M&N could request garbage truck axle loads

3. Evaluation-Pavement and Decking

The pavement investigation was performed on September 5, 2013 with wharf staff (Britt H and assistant) to identify typical pavement sections and the condition of the support timber decking below. Pavement was removed along sections of the wharf in both the roadway and parking areas to expose the condition of the timber decking below.

Condition

The overall condition of the pavement on the wharf is poor. There is pavement cracking running parallel to the deck boards (reflective cracking) throughout. In many locations the cracks run in the orthogonal direction as well, creating loose pieces of asphalt concrete (AC). The AC was removed in a few locations around the wharf to examine the condition of the timber decking below. The decking appeared to be in fair condition below the AC, except in some locations where there was splitting and rotting of the timber boards. In these areas, there is a noticeable depression in the pavement in addition to the prevalent cracked condition of the pavement.   It is estimated that less than 20% of the existing pavement is in serviceable condition, further, it is located in isolated patched areas that would preclude being able to salvage these areas.

Evaluation

Based on the observed damage, it appears that the following process occurs:

- At first the new pavement is stiff across the boards.
- The decking connections are simple nail connections, and the boards are already very flexible. Trucks drive across, and over time loosen the nails and the area around the nails as the boards readily bend.
- As the nails loosen, the timber decking becomes more flexible and more movement occurs.
- At the same time, the pavement crack initiates. This allows more movement, and more cracks appear. The more cracks in the AC allow more flexibility in the timber decking. The more flexible the timber decking, the more cracks appear in the AC.
- The cracks also allow water seepage onto the timber decking. A small check in the board becomes rotted and the rot exponentially increases.
- The AC cracks rapidly increase as the timber decking flexes more and more.
- Also, when all the decking boards have butt joints in the same place, this leads to longitudinal cracks in the road at the butt joint location, as there is the most movement at the end of each deck board.

Photos

The following Photos show examples of the pavement condition across the wharf, going north to south:

- Photo 9 through Photo 15 show views of the pavement (overall and a close-up of the deck boards) between Bents 50 and 100.
- Photo 16 shows an example of timber deck boards with aligned butt joints (around Bent 100).
- Photo 17 through Photo 25 show views of the pavement (overall and a close-up of the deck boards between Bents 100 and 150.

Potential Improvements

Some improvements can be made as part of the Wharf Master Plan. Some preliminary suggestions are listed below:

- Replace rotting and splitting deck boards
- Make sure deck boards alternate joint locations
- Add waterproofing layer between AC , or future pavement systems, and deck boards

Santa Cruz Wharf

Vehicle Loading Memorandum

Prepared for:

CITY OF
SANTA CRUZ

Prepared by:

moffatt & nichol

2185 North California Blvd, Suite 500
Walnut Creek, CA 94596

Jan 23, 2014

1. Summary

The purpose of this memorandum is to provide an engineering analysis of the Santa Cruz Wharf for the following:

- Master plan features (new east walkway)
- Vehicle capacity-existing Wharf
- Vehicle capacity increase-upgrades

The maximum allowable vehicle weight on the Wharf roadway is dependent on the capacity of the existing roadway which is governed by stringer spacing and the condition of the decking. The analysis shows the allowable vehicle axle load the Wharf can support (capacity) is 29,000 to 34,400 lbs. The range of axle loads (demand) of Santa Cruz Fire Dept. (FD) trucks is 23,000 to 31,000 lbs. Therefore, some of the FD trucks fall within the allowable range of the existing wharf. With improvements to the existing wharf, the maximum allowable axle would be 35,600 lb., which would include all the FD trucks. These allowable axle loads correspond to the typical 18-21 ton truck (80% total truck weight is on rear axle based on the American Association of State Highway and Transportation Officials (AASHTO)).

The maximum allowable vehicle on the Wharf parking areas is an axle load of 13,500 lbs corresponding to an 8 ton truck, less than half what the roadway can support. This capacity is sufficient for passenger cars and light trucks but general trucks should not enter these areas.

A FD truck turning radius analysis was performed; some of the FD trucks will be able to turn around within the existing roadway (without having to back up) at the circle at the end of the wharf and all can turn through the east parking lot.

Moffatt & Nichol (M&N) recommends:

1. The City review the operations plan for their vehicles, especially emergency FD trucks, that need to access the wharf
2. Provide barriers or clear markings to restrict trucks from portions of the wharf with reduced load capacity (i.e. parking spaces)
3. Test the existing wharf deck boards and stringers to determine actual bending and shear stress values that may increase the allowable vehicle weight.
4. Retrofit turnaround areas to increase the capacity to match the straightaway road capacity.
5. Design the new pedestrian walkway (promenade) for a minimum capacity of 36,000 lbs axle load.

2. Introduction

Moffatt & Nichol (M&N) performed onsite observations of the condition of the Santa Cruz Wharf structure and an analysis for the design of the primary structural elements of the new pedestrian walkway expansion on the east portion of the Wharf. The configuration of the new structure will be similar to the existing wharf (pile bent spacing, cap size and type, stringer size and spacing) and will have a capacity of at least the capacity of the existing wharf roadways. The capacity of the Wharf roadway, where emergency vehicles will travel, (at turnaround and straightaways) were calculated and compared with the loads (weights) of the larger vehicles—fire trucks. Recommendations for the new structure and retrofit to the existing structure are presented at the end.

2.1 Scope

- Determine load bearing capacity of existing wharf for design vehicles.
  - Due to varying stringer size and spacing, perform calculations for 4x12 stringers at 12-inch spacing and 6x12 stringers at 16-inch spacing.
  - Look at trucks driving on the straightaways (parallel to roadway and stringers)
  - Look at trucks driving at turnaround areas (perpendicular to stringers)
- Recommend structural member size and spacing for new promenade.
- Determine recommendations for retrofitting existing wharf structure to accommodate design vehicles.

2.2 Description of Structure

The Santa Cruz Wharf was originally constructed in 1914, and has been widened along the length during its history. The wharf is of timber construction, with 183 rows (bents) of vertical timber piles that vary in age from 2 – 99 years. These piles are the Wharf foundation and provide the primary resistance to lateral forces (waves, earthquake, vessels docking) acting as cantilever elements embedded 20 ft into the sandy bottom. In addition to the 4,700 (approx.) total vertical piles, there are batter (slanted) piles scattered throughout the wharf that provide some additional lateral stiffness. The piles support transverse 12x12 beams (caps), longitudinal 4x12 and 6x12 beams (stringers), and 3x12 decking topped with 1-3 inches of asphalt paving. The stringer size and spacing (on centers) are the main factors determining the allowable vehicle capacity and varies as follows:

- Roadways:
  - 4x12 at 10-12 inches and
  - 6x12 at 16 inches
- Parking areas and under buildings: 4x12 at 24 inches
- Repairs and replacements by wharf maintenance crew: 6x12 at 16 inches

There are seven distinctive areas of the wharf structure:

1. Trestle
2. Trestle with Parking
3. Wharf
4. Wharf East Parking
5. Angle
6. Stagnaro's
7. End of Wharf

Figure 1 and 2 show these features of the wharf structure described above. The arrangement of the caps, stringers, and the connections is varied, particularly along joints of different construction periods of the wharf.

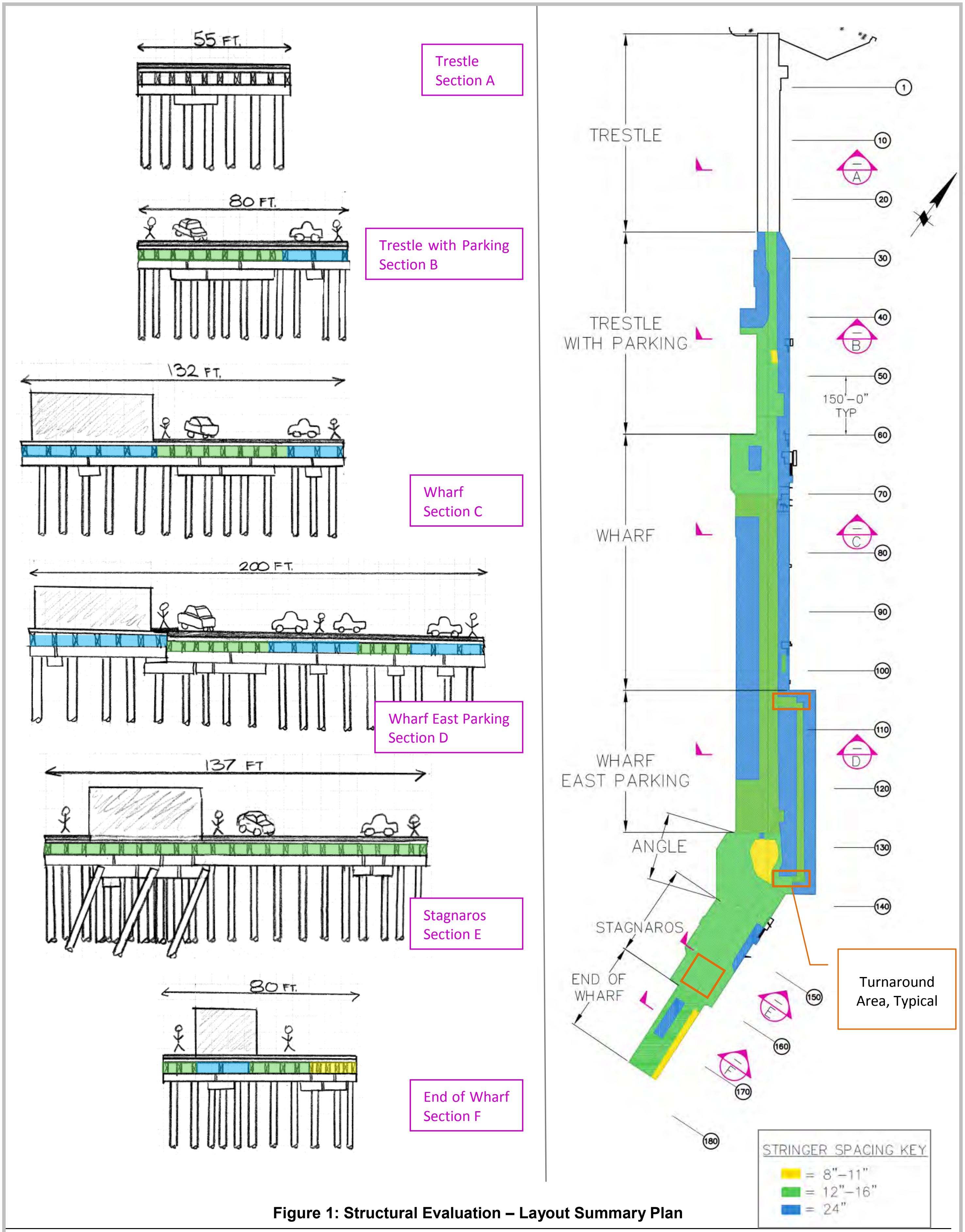


Figure 1: Structural Evaluation – Layout Summary Plan

2.3 Methodology

The analysis of the structural elements was performed according to National Design Specification for Wood Construction (NDS 2005) guidelines, the California Building Code (CBC 2010), and AASHTO Bridge Design Specifications 2010.

- Load Combinations are per CBC 2010 for Allowable Stress Design:
    - Case 1: Dead + Live
    - Case 2: Dead + Truck Live
- Distributed live load is 100 psf, per CBC 2010, Table 1607.1
- Member Sizes for new construction are assumed to be the same as the existing wharf for consistency:
    - Caps are 12x12
    - Stringers are 4x12
    - Decking is 3x12
    - Asphalt (AC) covering is 2 inches
    - Analysis was also performed for a 6x12 stringer
- For analysis, the representative stringers under the roadway are:
    - 4x12 at 12 inches on center
    - 6x12 at 16 inches on center
- Member sizes are dimension lumber (i.e. 3x12 dimension is 3.0 x 12.0 inches as opposed to nominal size that actually measures 2.5 x 11.5 inches)
- The capacity of the timber structure was based on the assumption that the timber is Douglas Fir Larch (North) Select Structural.
- In analyzing the existing roadway, the decking capacity was multiplied by a 0.75 factor to allow for variability in the present condition (based upon observations, and Table 2-1 of FEMA 356. Note: when testing of the samples of the deck boards is performed as part of the engineering report, this value may be recalculated).
- Bent spacing is 15 ft. on center.
- For the truck loading, the capacity is increased by 1.6 based on the increased NDS load duration factor for ten minute load duration. No impact load factor is used, as the traffic is slow on the wharf.
- Per Table 2, the widths of the truck are between 112 and 120 inches mirror to mirror (approximately 9’-6”). The axle width is conservatively assumed to be 6 ft. from tire centerline to tire centerline (AASHTO Bridge Design Specifications).
- Wheel width is 20 inches (AASHTO Bridge Design Specifications, 2007 section 3.6.1.2.5)

- For the cantilever end at new promenade, centerline of furthest truck tire is assumed to be a minimum of 2 ft. from the edge of the wharf.

Calculations were performed using a spreadsheet and this methodology to compare the member capacities to the imposed demand (vehicle load). The resulting demand-to-capacity ratio (DCRs) is acceptable if less than 1.0, or indicates overloading of the structure if greater than 1.0.

The analysis primarily addresses the structural capacity to support vehicle loads, but another consideration is the resultant deflection of the structure. The deflection of the supporting structure (underlying decking and stringers) has a significant effect on the condition and durability of the asphalt surfacing which has been a very large ongoing maintenance task at the wharf. Even though the deck may support a large vehicle, the resulting deflection may be to the point it causes the pavement to severely crack and ravel (break up). The analysis of the pavement is covered in a separate memorandum.

3. Condition Assessment

Based on the inspection, the condition of the wharf structural members are in fair-good condition. There are isolated areas of rot on stringers and caps. The decking was found to be soft or splintered in various locations throughout the wharf, and therefore the 0.75 reduction factor was applied to the capacities of the existing decking per Section 2.3. A sketch of a typical section is shown in Figure 2.

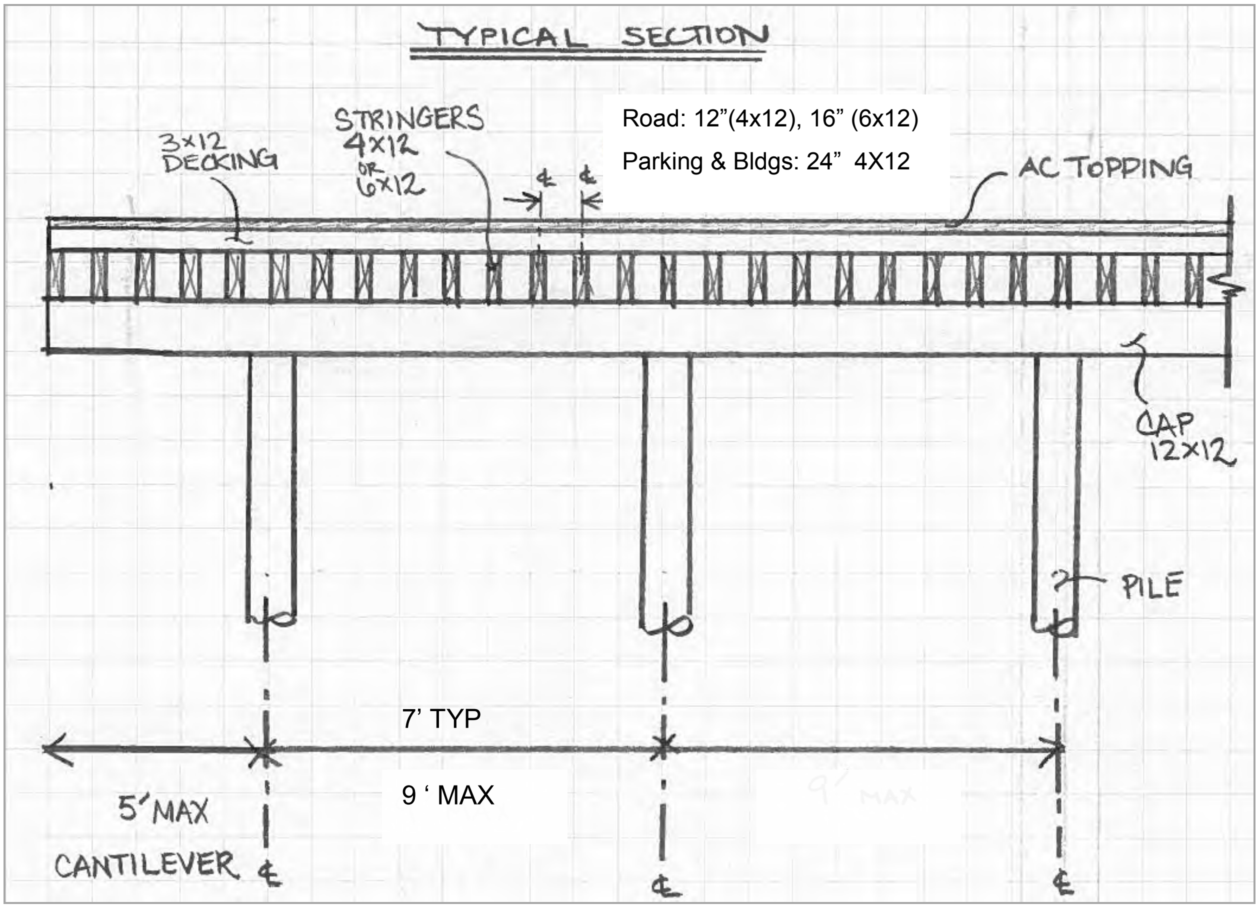


Figure 2: Sketch of Typical Section

4. Analysis

4.1 Load Cases

The following three load cases were examined:

- Case 1: Dead + Uniform Live Load
- Case 2A: Dead + Vehicle Live (Truck Parallel to Road/ Stringers)
- Case 2B: Dead + Vehicle Live (Truck Perpendicular to Road /Stringers)

Figure 3 presents a visual summary of the cases. Cases 2A and 2B always governed over Case 1, therefore most of the focus was on Cases 2A and 2B. The locations of the turnarounds can be seen in Figure 1.

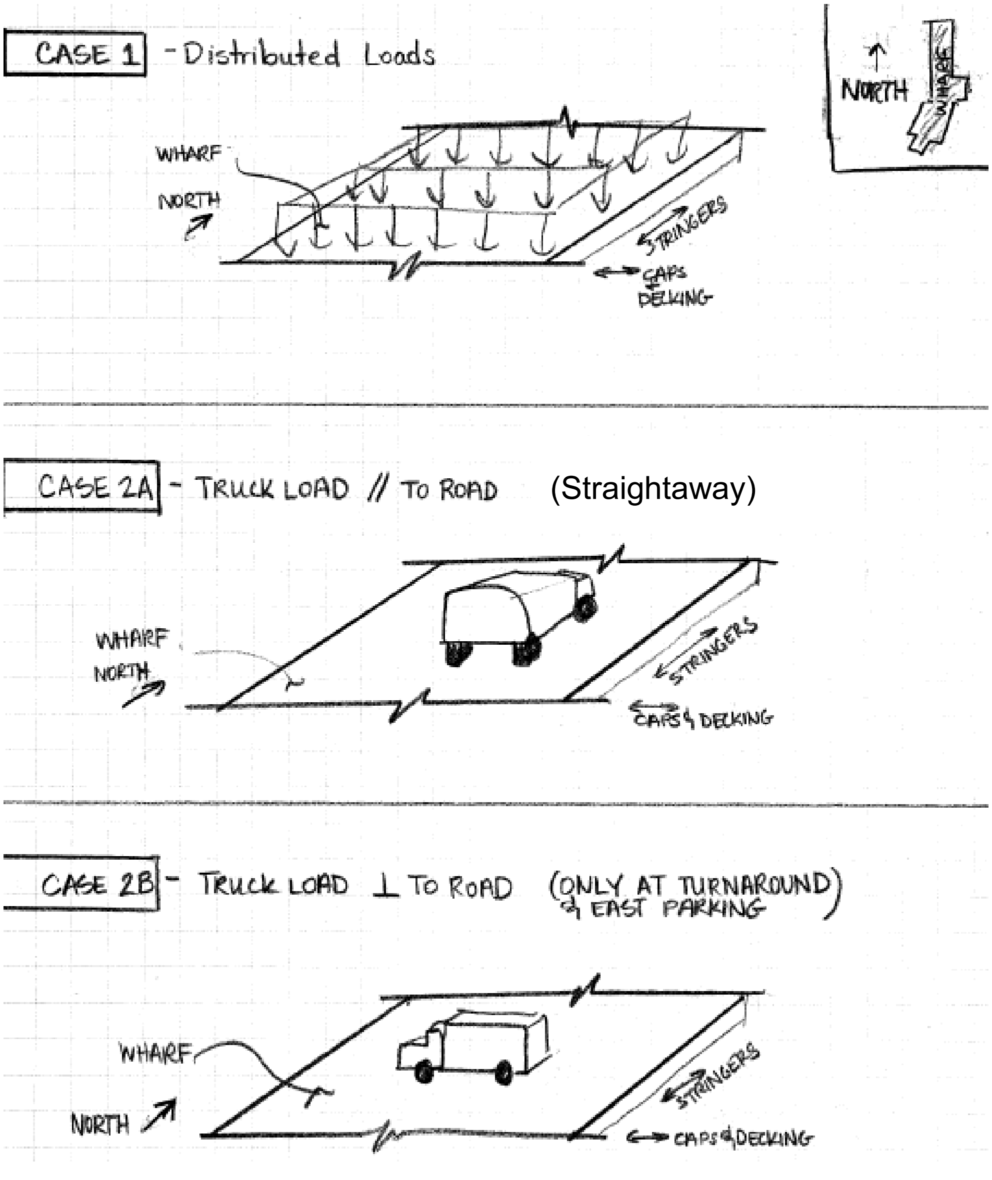


Figure 3: Summary of Cases

4.2 Member Capacities

The individual member capacities are summarized in Table 1. The values below were increased by the Load Duration Factor of 1.6 for temporary truck loading.

Table 1: Structural Member Capacities

Structural Member	Allowable Bending Moment	Allowable Shear Force	Allowable Axial Force
	lb-ft	lb	lb
Cap, 12x12	61440	26080	-
Stringer, 4x12	18560	8800	-
Stringer, 6x12	30720	12960	47000 compr/ 109400 ten
Decking, 3x12	4160	6560	-
Pile, 12" Diam.	42560	15520	65600

4.3 Design Vehicles

The largest emergency vehicles accessing the wharf are fire trucks. Information on the fire truck fleet was received from the Santa Cruz Fire Department; shown on Table 2 .

Table 2: Fire Truck Information

Apparatus	GAWR (Gross Axle Weight Rating)		Wheel Base (inches)	Mirror-to-Mirror Width (inches)	Actual Measured Weights				
	Steering Axles (lb)	Drive Axle (lb)			Front Axle Left (lb)	Front Axle Right (lb)	Rear Axle Left (lb)	Rear Axle Right (lb)	Total Weight (lb)
E3110/775	22,800	24,000	185	115	*	*	*	*	*
E3111/772	16,000	24,000	171	112	6,800	7,000	9,850	10,300	33,950
E3112/777	22,800	24,000	185	115	*	*	*	*	*
E3113/773	16,000	24,000	175	112	7,000	7,500	10,450	10,450	35,400
E3114/771	22,000	31,000	217	120	9,400	9,400	14,100	14,300	47,200
T3170/778	22,800	31,000	160	115	*	*	*	*	*
E2710	22,800	24,000	185	115	*	*	*	*	*
E2730	12,000	23,000	176	118	*	*	*	*	*

* No value available

In addition, information for the municipal garbage trucks that regularly access the wharf was provided by the City of Santa Cruz (the City). The heaviest measured truck was 23.3 tons. These trucks have dual rear axles with dual tires. No information about the weight distribution between the front and rear axles was available, however the distribution of axle loads for a typical truck is 80% of the total weight on the rear axle and 20% on the front axle (HS designation per AASHTO Bridge Design Specifications) resulting in 37,300 lbs on the rear axles. Although the garbage truck rear (dual) axle load is heavier overall, the FD trucks produce a larger load on the stringers due to the single rear axle.

The drive axles of the Fire Department fleet range from 23,000 to 31,000 lbs. These loads are comparable to a general truck weighing 17.5 to 27 tons using the AASHTO distribution of 80% on the rear axle. These values can be used to set a limit for general trucks that might drive on the new walkway.

Vehicle Load Capacities

The analysis was performed for a vehicle traveling along the straight roadway, parallel to stringers, (Case 2A) and then for vehicle perpendicular to the stringers (Case 2B) as when turning around-see Figure 1. Table 3 presents the governing demand-to-capacity ratios for the structural members and the maximum allowable point loads based on Case 2A and Table 4 shows the values for Case 2B. The 0.75 reduction factor for the existing decking is not applied at this point in the calculation. The maximum allowable axle load are higher for Case 2A than Case 2B. The governing cases (shown highlighted) are primarily the stringers in bending, with the cap in shear for the 6x12's in Case 2A.

Table 3: Case 2A - Structural Member Demand-to-Capacity Ratios

Structure	DEMAND (lb) Maximum Allowable Point Load* (1/2 x Axle Weight)	Demand-to-Capacity Ratios					
		Decking		Stringers		Cap	
		Shear	Moment	Shear	Moment	Shear	Moment
4x12 Stringers @ 12 in.	18,500	0.30	0.71	0.51	1.00	0.88	0.65
6x12 Stringers @ 16 in.	21,200	0.40	0.77	0.55	0.93	1.00	0.73

* Allowable loads are before 0.75 reduction factor has been applied to decking

Table 4: Case 2B – Structural Member Demand-to-Capacity Ratios

Structure	DEMAND (lb) Maximum Allowable Point Load* (1/2 x Axle Weight)	Demand-to-Capacity Ratios					
		Decking		Stringers		Cap	
		Shear	Moment	Shear	Moment	Shear	Moment
4x12 Stringers @ 12 in.	14,500	0.30	0.63	0.60	1.00	Case 2A governs	
6x12 Stringers @ 16 in.	17,800	0.36	0.77	0.68	1.00		

* Allowable loads are before 0.75 reduction factor has been applied to decking

Table 5 shows the allowable truck sizes for Case 2A and 2B with the 0.75 decking reduction factor (existing wharf condition) and without (retrofit wharf condition). The decking governed for the 6x12 stringers.

Table 5: Wharf Vehicle Axle Capacities for Case 2A and 2B

Existing Wharf

Structure	**Demand** (Fire Truck Axle Weight Range in lb)	Case 2A (Truck on straightaway)		Case 2B (Truck at turnaround)	
		Capacity (One Axle in lb)	**Maximum General Truck Size in tons**	**Capacity** (One Axle in lb)	**Maximum General Truck Size in tons**
4x12 Stringers @ 12 in.	23,000 - 31,000	37,000	23.0	29,000	18.0
6x12 Stringers @ 16 in.		41,000	25.5	34,400	21.5

Retrofit Wharf

Structure	**Demand** (Fire Truck Axle Weight Range in lb)	Case 2A (Truck on straightaway)		Case 2B (Truck at turnaround)	
		Capacity (One Axle in lb)	**Maximum General Truck Size in tons**	**Capacity** (One Axle in lb)	**Maximum General Truck Size in tons**
4x12 Stringers @ 12 in.	23,000 - 31,000	Same as for Existing Wharf			
6x12 Stringers @ 16 in.		42,400	26.5	35,600	22.0

4.4 Deflections of Stringers

Preliminary calculations were performed for the deflections of the stringers under the live load (only) for the three load cases. The results are presented in Table 6.

Table 6: Deflections of Stringers

Case	4x12@12		6x12@16	
	Axle Load	Deflection (in)	Axle Load	Deflection (in)
1	-	0.10	-	0.11
2A	37,000	0.51	42,400	0.63
2B	29,000	0.64	35,600	0.84

As mentioned above, this deflection has a significant effect on the pavement condition and life. The deflections due to the larger trucks, particularly those that access the wharf on a regular, recurring basis, for delivery and pick up, are the primary cause of asphalt deterioration. This topic is addressed further in the memorandum on Paving.

4.5 Truck Turning Radius

The Fire Department provided information about the turning radii of a few of the fleet trucks. This information is presented in Table 7.

Table 7: Fire Truck Turn Radii

Apparatus	Width required to Turn (perpendicular to roadway direction)	Length Required to Turn (in direction of roadway)
E3113/773	67.5 ft	38 ft
E3114/771	76 ft	48 ft
T3170/778	62 ft	40 ft

Sketches of the E3113 truck turning at the end of the wharf (area 1) and the east parking lot (area 2) were done in AutoCAD and are presented in Figure 4 and Figure 5. According to the Fire Department, the E3113 truck is more likely to be used than the larger E3114 truck. It appears that the truck can fit at both these locations. Although the East Parking lot location has more turning room, turning at the end of the wharf must be provided to access the entire length of the wharf and should be the first priority if increased capacity is required for the larger fire trucks.

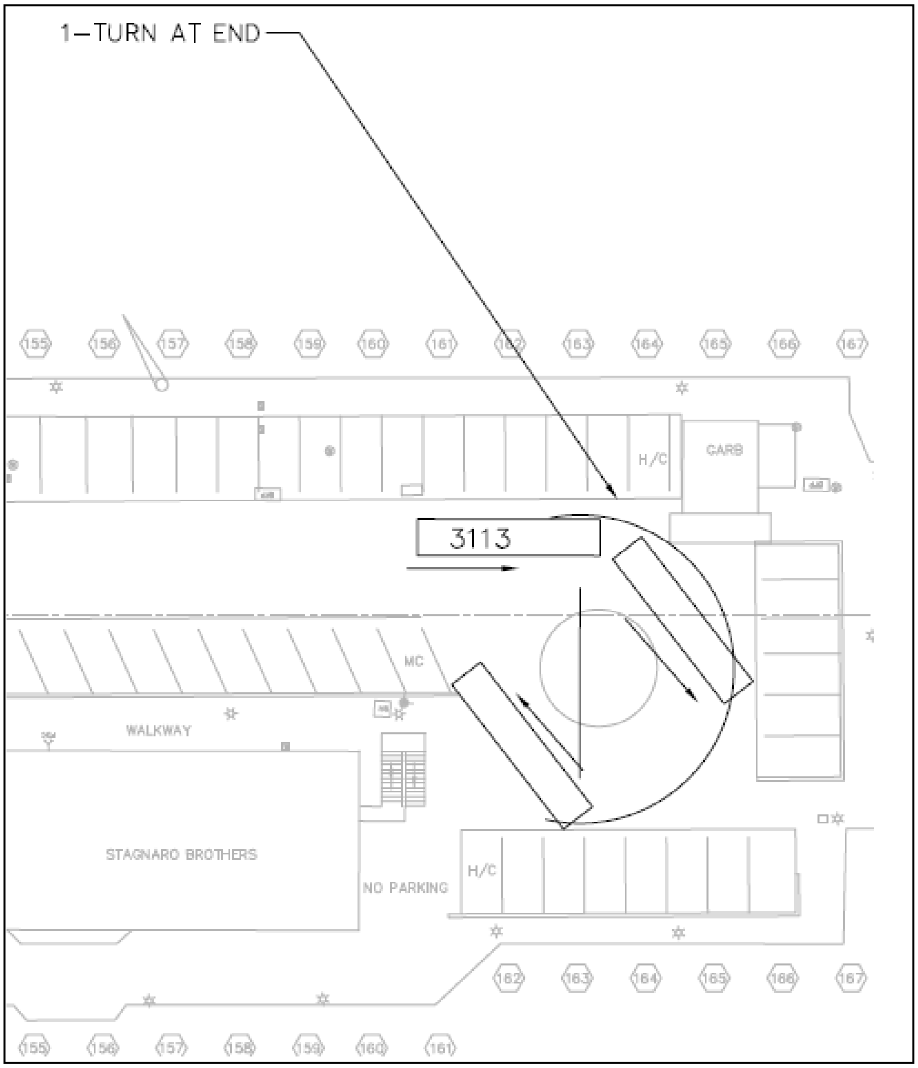


Figure 4: E3113 Fire Truck Turn at End of Wharf

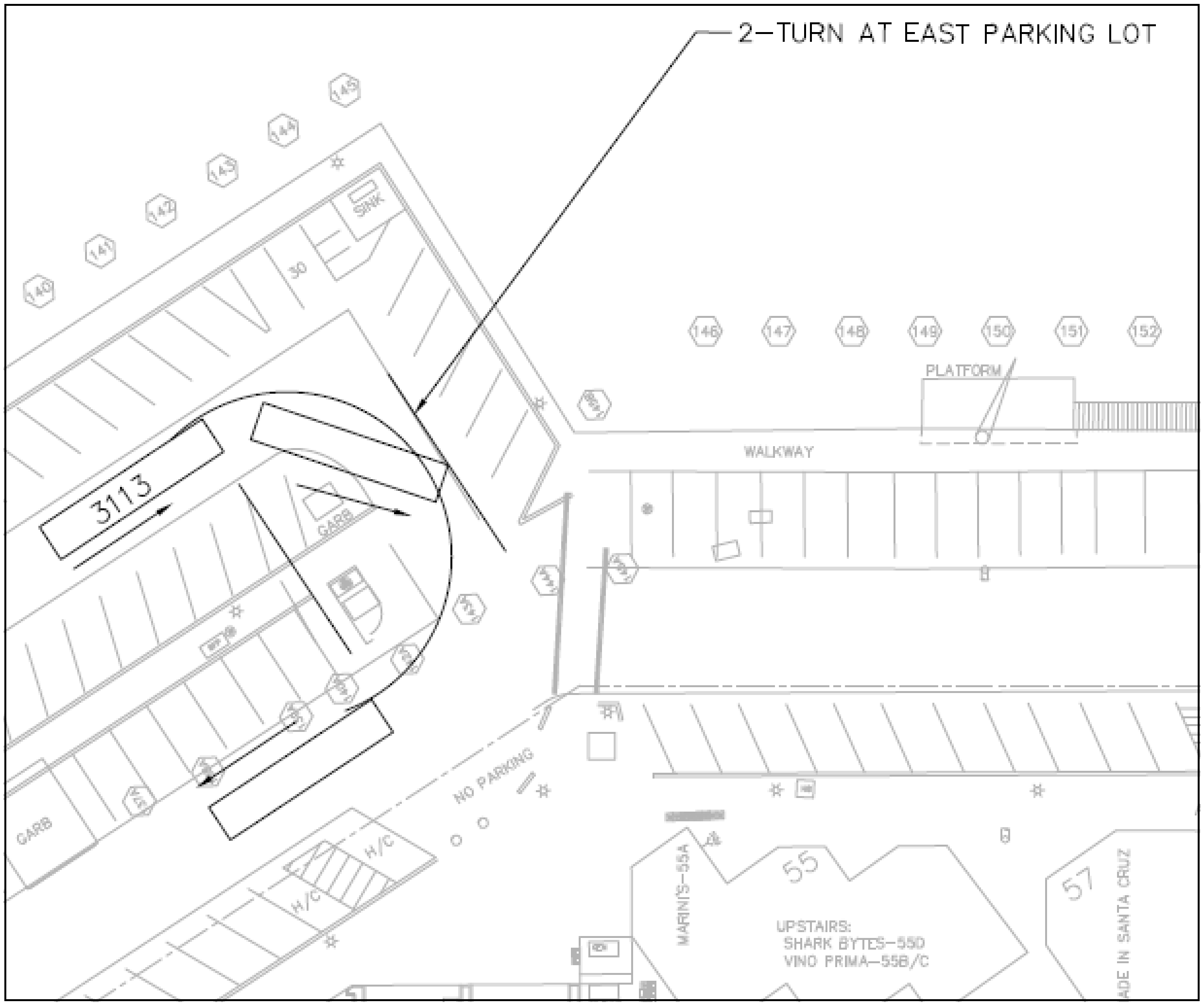


Figure 5: E3113 Fire Truck Turn at East Parking

5. Conclusions

The analysis shows that the allowable vehicle axle load the existing Wharf can support (capacity) is 29,000 to 34,400 lbs. The range of axle loads (demand) of Santa Cruz Fire Dept. (FD) trucks is 23,000 to 31,000 lbs; therefore some of the FD trucks are within the allowable range. With improvements to the wharf, the maximum allowable axle would be 35,600 lb., which would include all the FD trucks. These allowable axle loads correspond to the typical 18-21 ton truck (80% total truck weight is on rear axle based on the American Association of State Highway and Transportation Officials (AASHTO)).

The maximum allowable vehicle on the Wharf parking areas is an axle load of 13,500 lbs corresponding to an 8 ton truck, less than half what the roadway can support. This capacity is sufficient for passenger cars and light trucks but general trucks should not enter these areas.

The pavement deterioration that has been observed over time at the wharf is due in large part to the structural deflections caused by truck access onto the wharf. A separate memo addresses this more fully.

Some of the FD trucks are able to turn around within the existing roadway (without having to back up) at the circle at the end of the wharf and all can turn through the east parking lot.

In summary:

- The existing wharf roadway allowable capacity will support an axle load of 29,000 to 34,400 lbs, which corresponds to general trucks of 18 tons
- The existing wharf capacity is adequate for many of the Fire Department's trucks.
- The new walkway should be designed to support a minimum axle load of 36,000 lbs which would support all FD trucks
- The existing wharf capacity is limited primarily by:
  - The 4x12 stringers still in place
  - The stringers in the turnaround areas

6. Recommendations

1. The City review the operations plan for their vehicles, especially emergency FD trucks, that need to access the wharf
2. Provide barriers or clear markings to restrict trucks from portions of the wharf with reduced load capacity (i.e. parking spaces)
3. Test the existing wharf deck boards and stringers to determine actual bending and shear stress values that may increase the allowable vehicle weight.
4. Retrofit turnaround areas to match the straightaway road capacity.
5. Design the new pedestrian walkway (promenade) for a minimum capacity of 36,000 lbs axle load with the following structural members:

Pile Cap:	12x12 timber
Stringers:	6x12 @ 18 inch timber
Decking:	3x6 timber
Asphalt thickness:	2-9 inches depending on pavement design
Pile spacing:	7 ft on center typical, and up to 9 ft. maximum

3.5 Conclusions and Recommendations

1. Pavement should be replaced across the entire wharf.
    - Install a test section to determine most effective asphalt system (plywood under layer Alt 2 or grid reinforcement Alt 2.1)
    - Install selected system (Alt 2 or 2.1) with waterproofing layer between AC and deck boards to minimize cracking.
    - Alternate deck board joint locations
    - Consider rubberized asphalt in place of conventional asphalt.
    - Install by phases in the areas shown on Figure 3-6
2. Install drain inlets in vehicle area to treat runoff with media filtration to address water quality
3. Pursue grant funding for water quality improvement associated with the repavement of the Wharf
4. Limit truck traffic to the greatest extent possible to minimize damage. This may include the following:
    - Replace garbage collection system with onshore vacuum collection to eliminate garbage truck traffic onto wharf
    - Consider use of smaller, light weight collection vehicles
    - Designate smallest effective Fire Department truck to go onto wharf
    - Require delivery trucks of smallest practical size, or require that deliveries be made at the end of the route when the truck weight is presumably at a minimum.

Attachment 3A Photographs

Photo 3-1: Pavement near Lifeguard Station

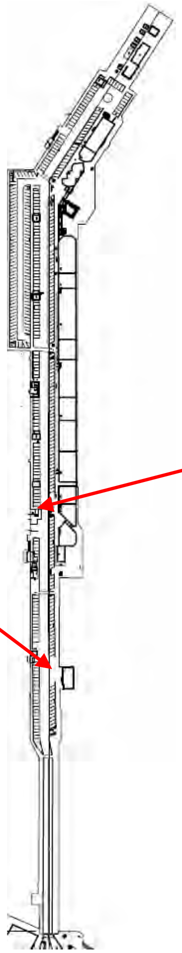


Photo 3-2: Pavement near Bonnie's

Photo 3-3: Pavement near Lifeguard Station

Photo 3-4: Pavement near Bonnie's

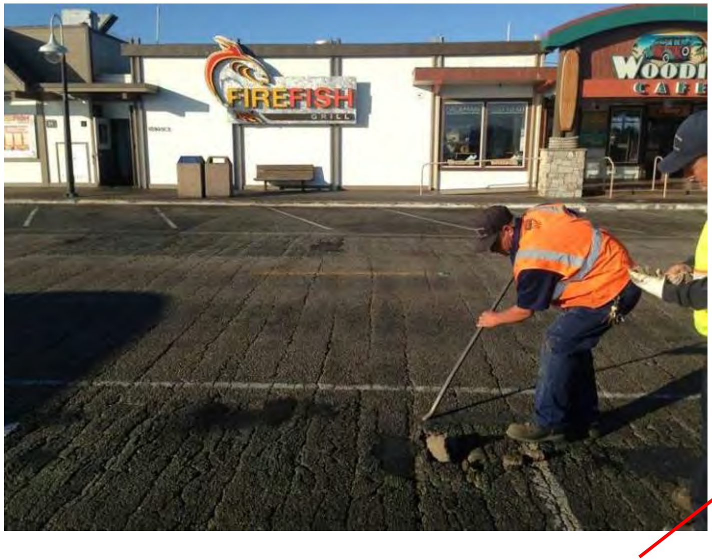


Photo 3-5: Pavement near Firefish

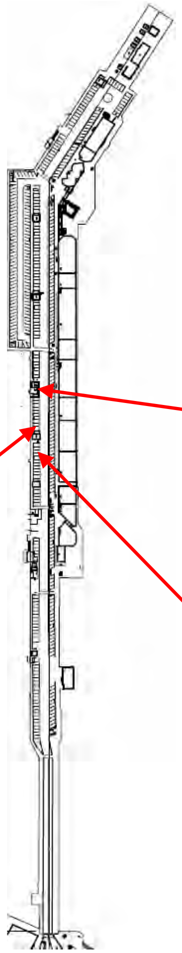


Photo 3-6: Pavement near Firefish and Riva with Large Truck

Photo 3-7: Pavement near Firefish

Photo 3-8: Decking with Aligned Butt Joints (near Firefish/ Woodie's)

Photo 3-9: Pavement near Riva

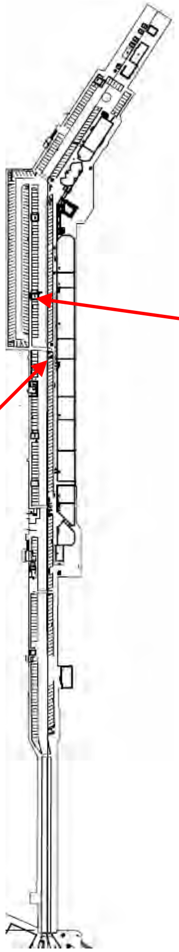


Photo 3-10: Pavement near Miramar

Photo 3-11: Pavement near Riva

Photo 3-12: Pavement near Miramar (showing loosening nails)

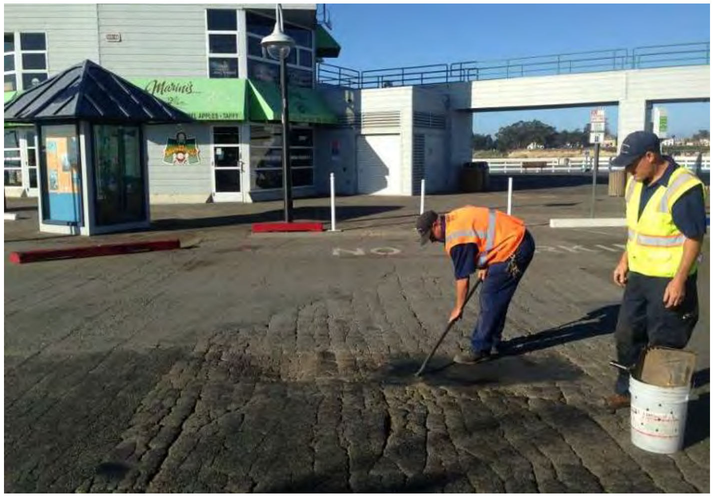


Photo 3-13: Pavement near Marini's

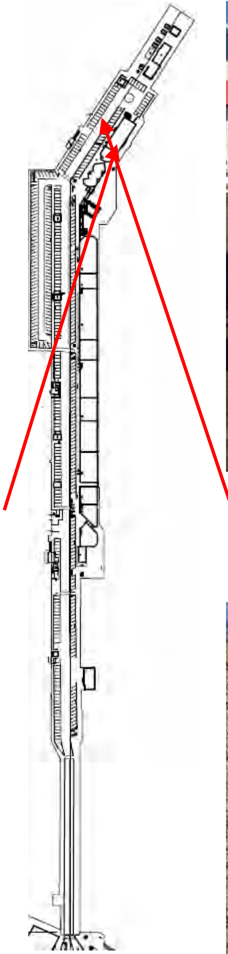


Photo 3-14: Pavement near Marini's and Made In Santa Cruz

Photo 3-15: Pavement near Marini's

Photo 3-16: Pavement near Marini's and Made In Santa Cruz

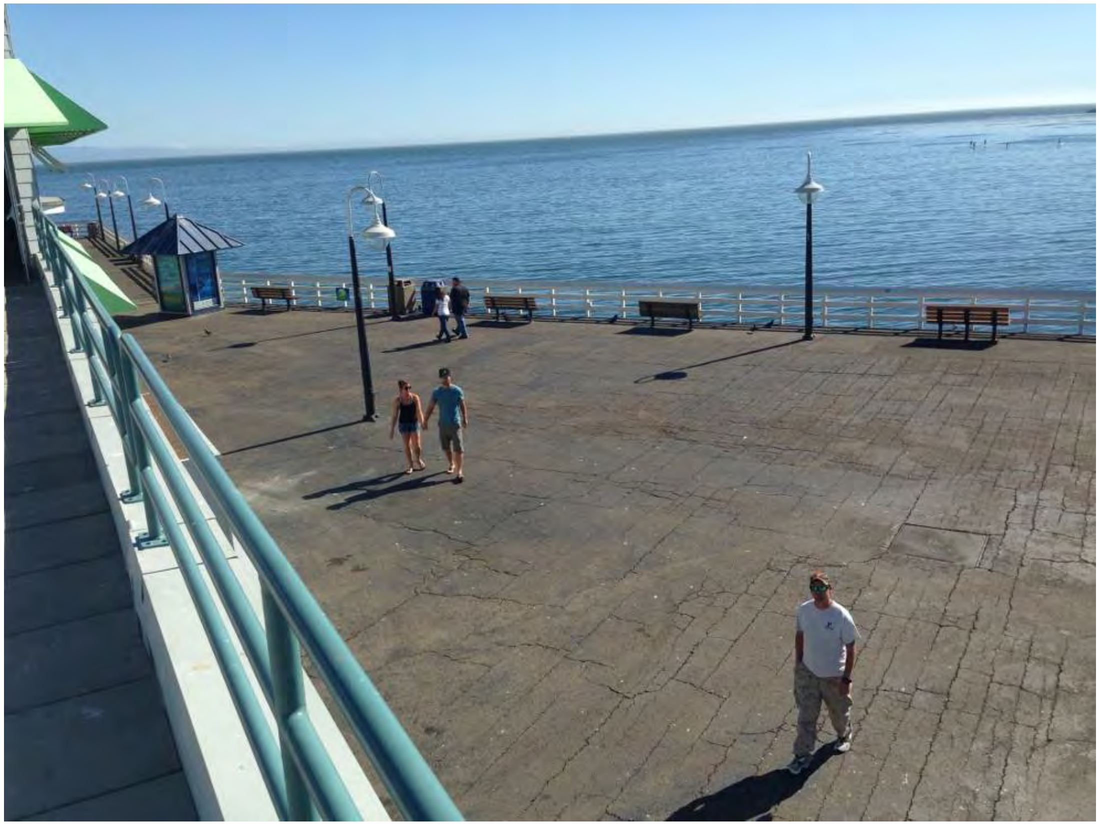


Photo 3-17: Pavement on West side of wharf near Marini's

Attachment 3B Memoranda

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MEMORANDUM

To: **Norm Daly, Jon Bombaci, City of Santa Cruz**

From: **Brad Porter, PE**

Date: **Feb 18, 2014**

Subject: **Santa Cruz Wharf Pavement Alternative**

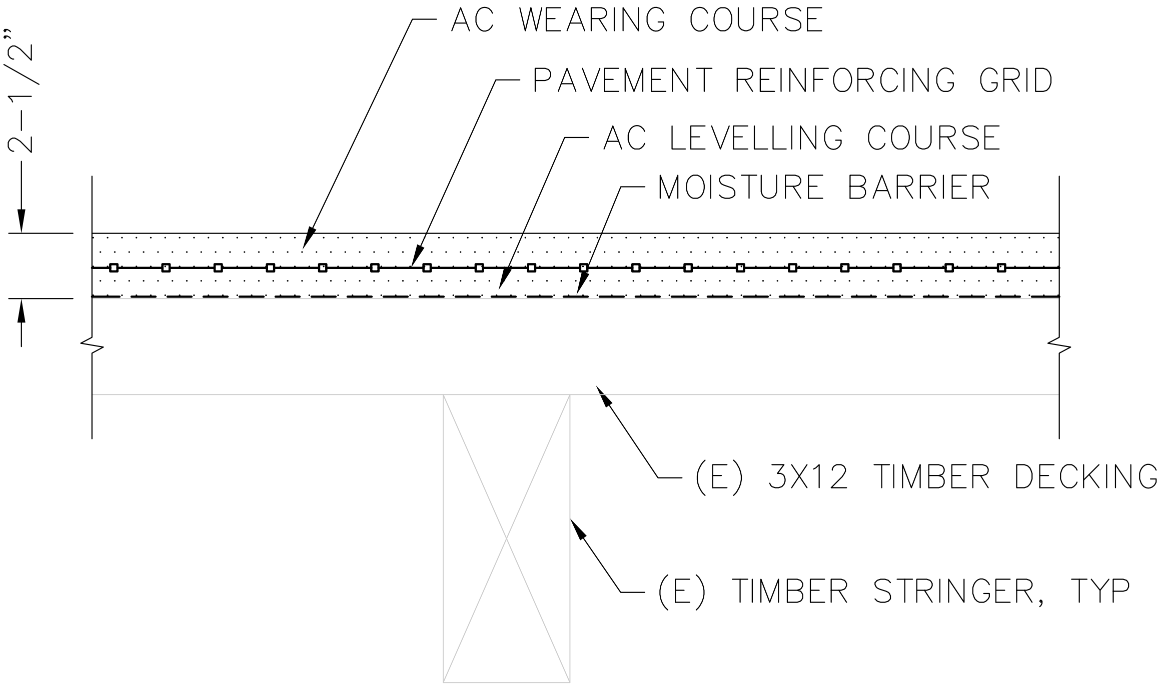
M&N Job No.: **8181**

The purpose of this memo is to describe a pavement system that was used that may have application at Santa Cruz Wharf. The Hyde Street Pier in San Francisco is a similar timber structure to Santa Cruz Wharf and has asphalt surfacing over the timber decking. It had experienced significant reflective cracking over time, as is common for this type of structure. We performed an evaluation of the pier which included some alternatives to the resurfacing. The pier was resurfaced in 2001 based upon the grid reinforced pavement shown on attachment 1. The specifications and some details for that project:

- Pavement Reinforcing Grid: GLASSGRID 8501, and a fiber glass mesh grid system -placed between the 1" leveling course of asphalt, and the 1 1/2" wear course asphalt (http://www.tensarcorp.com/Systems-and-Products/GlasGrid-Pavement-Reinforcement-System)
- Moisture Barrier: Petrotac, a rubberized asphalt membrane to lay on top of the wooden decking (http://geotextile.com/product/petromat.html)
- This project was done in the summer of 2001
- The system has held up well.

The pavement has held up well over the past 13 years. The Hyde St Pier is subject to lighter loading than Santa Cruz Wharf--it is open to the public for foot traffic only; pickup truck size vehicles go on the pier occasionally. Attachment 2 shows photographs taken in 2000 and comparable photos taken in 2011 when the new pavement was 10 years old. The pavement has held up well; there are some minor reflective cracks but overall it is in very good condition. It should be noted that prior to the resurfacing of Hyde St Pier the reflective cracking observed is similar to that observed in the Commons on Santa Cruz Wharf which has similar pedestrian loading only.

Attachments: 1-Pavement Figure; 2-Photographs Before and After



SECTION

NTS

MOFFATT & NICHOL
ENGINEERS

WALNUT CREEK, CALIFORNIA

HYDE STREET PIER

FIGURE 3
PROPOSED AC REPAIR

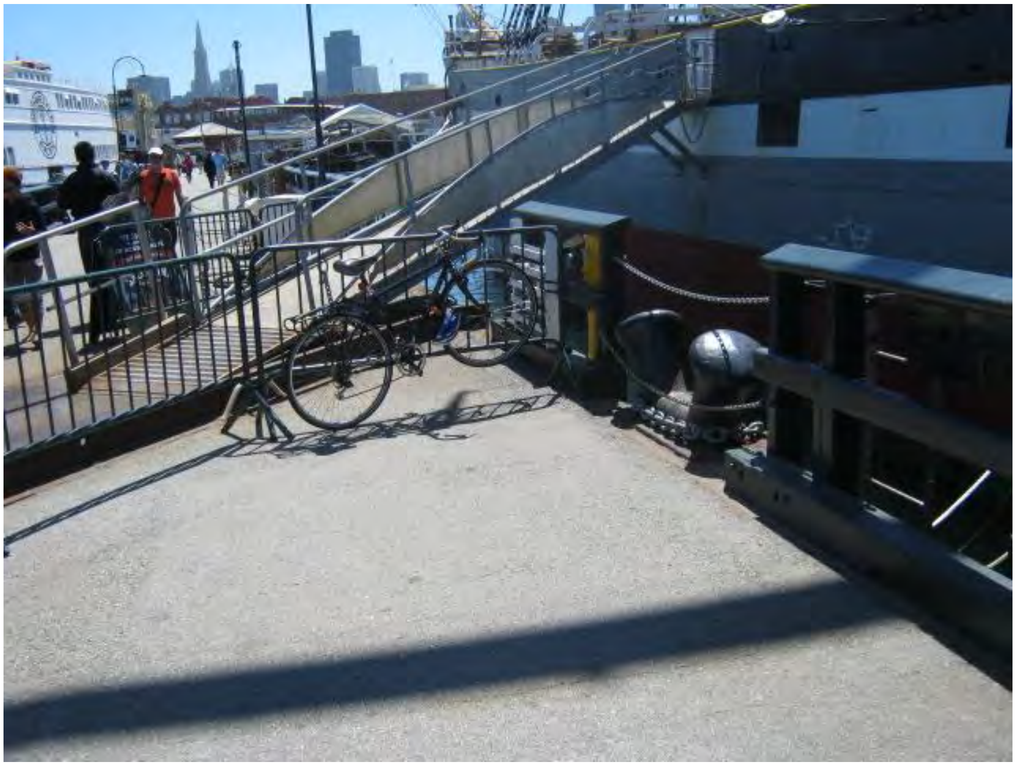
1a-Looking west at Bent 3- 2001

1b-Looking north near Bent 3--2011.

2a-Looking west in the vicinity of Bent 17-2000

2b-Looking west in the vicinity of Bent 17.-2011

3a-Looking south from north end of pier.-2000



3b-Looking south from north end of pier.-2011

3c-Looking south from north end of pier.-2011

4-Minor Reflective Crack-2011

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MEMORANDUM

To: **Norm Daly, Jon Bombaci, and Mark Dettle--City of Santa Cruz**

From: **Brad Porter, PE**

Date: **March 5, 2014**

Subject: **Santa Cruz Wharf Pavement Comments, Response, Recommendations**

M&N Job No.: **8181**

The purpose of this memo is to present the results of further study and analysis that we have performed in response to comments to our previous memo of January 23rd regarding vehicle loads and paving on Santa Cruz Wharf. A summary of these comments and our responses and recommendations are presented below.

Comment Summary

(For full text see email of Jan 30 from Mark Dettle, PE Dir PW)

1. Analysis of added stiffness due to plywood
2. Pavement cracks in pedestrian only areas (no truck travel, waves and flexible structure are contributing factors.)

(For full text see email of Feb 11 from Jon Bombaci, Wharf Supervisor,)

3. Plywood underlayment versus grid reinforced AC pavement
4. Hardwood decking versus concrete on walkways

Response to comments (corresponding numbers):

1. **Analysis of stiffnes**s -For the effect on the pile behavior, the addition of plywood would have no measurable effect. Adding plywood would only stiffen the deck in its plane causing the deck to act more as a rigid diaphragm. Since the connection to the piles is a pin connection, a more rigid diaphragm would cause no or very little change in the reaction to the pile. It would still be shear and bearing with little moment transfer. If there were some increased moment capacity, it would only reduce the moment in the piles due to double curvature rather than single for the cantilever pile.

2. **Pavement cracks in pedestrian only areas** – We agree there is enough movement in the existing timber wharf structure due to non-vehicular environmental loading to cause cracking along the gaps in the deck planks (outside of the vehicle areas). This effect was seen at Hyde St Pier as well (see photo in Feb 18 memo). It doesn't appear pavement cracking in areas outside the road way is to the same degree as we don't see raveling of the pavement in these pedestrian only areas as we do in the road way. We understand the Commons by Marini's was constructed in 1982 and the pavement would therefore be over 30 years old, unless it has been repaved.

   Recommendation: In the pedestrian only areas, where asphalt pavement is desired, utilize the grid reinforced asphalt described in our Feb 18 memo to minimize cracking.

3. **Plywood underlayment versus grid reinforced AC pavement** - Pavement on a flexible (timber wharf) structure is inherently subject to crack inducing forces from differential movement in the deck boards ("subgrade") due to bending from vertical (vehicle) loads and shear from horizontal (wave) load. Measures to limit these crack inducing forces include:

   a. Modify the structure to limit differential movement

   b. Increase pavement thickness to spread the load

   c. Increase pavement strength by increasing compaction and adding reinforcement

   d. Increase pavement resiliency through the use of additives to the asphalt/aggregate mix, such as rubberized asphalt

   We propose plywood underlayment as the most effective way of reducing the reflective cracking caused by the structure differential deflections occurring between deck planks under vehicular (vertical) and wave (horizontal) loading. While the City experienced problems with a previous plywood underlayment installation, it is our opinion the previous performance problems may have resulted from lack of expansion gaps, insufficient fasteners or other details in the constructed system. The plywood underlayment system we propose is designed detailed and will be constructed in such a way as to obviate the performance problems the City experienced previously. Further, use of a rubberized asphalt, properly compacted with a minimum thickness of 3 inches (varies from 3-6 inches for drainage) will increase the strength of the section. Accordingly, it is our opinion the proposed plywood underlayment system will address the concerns expressed.

   Furthermore, it is our opinion the alternative grid reinforced, AC pavement (see memo Feb 18 for full text) will not prove as effective over the life of the pavement at resisting the reflective cracking caused by the differential deflections occurring between deck planks particularly under vehicular loading. While there is no doubt the alternative AC pavement will perform better than the present unreinforced AC pavement, it is our opinion the performance and service life will be less than the proposed plywood underlayment system and the total cost of ownership may be greater than the plywood system. With regards to initial construction costs only, we offer the following regarding the plywood underlayment and grid reinforced alternatives for comparison:

e. Plywood Underlayment AC Pavement System
    i. Cost: $27/sf
    ii. Advantages: Bridges gaps in timber, provides added rigidity to pavement subgrade, will reduce deflection induced cracking, longer service life for both the pavement system and the wood decking substructure
    iii. Disadvantages: Initial Cost, added construction complexity (see also comment 1. Email)

f. Grid Reinforced AC Pavement System
    i. Cost: $14/sf
    ii. Advantages: Ease of construction, lower initial cost
    iii. Disadvantages: More flexible pavement system, differential deflection cracking will still occur , reduced service life when compared to plywood underlayment system

Recommendation: There are significant cost differences and construction challenges between these two alternatives. The initial cost of the plywood system is twice the grid reinforced alternative but will extend the useful service life of the paving system and protect the wood deck substructure far longer. While we recommend the plywood underlayment system, we believe it prudent to further validate the life cycle costs of these systems. Accordingly, we recommend a test section of each alternative be constructed along the main road way and monitor over time to evaluate cost and performance of each system.

4. **Hardwood Decking versus Concrete on Walkways**- There are two general areas where this choice arises. First, the pedestrian promenade being added along the east side of the main wharf and, second, between the pedestrian sidewalk between parking areas and the existing retail/restaurant buildings.

First, for the pedestrian promenade, the proposed open wood deck system offers a rich and distinctive aesthetic that connects the user to the ocean and to the historic fabric of the original Santa Cruz wharf. It is interesting to note that at the Santa Monica Pier, which is a concrete structure, the decision was made to use a wood decking over the concrete slab for the marketing and aesthetic appeal that it offered to the visitor. Technically, the other concerns can be addressed as follows: the hardwood proposed for use will not burn unless a sustained heat source is applied (e.g. a blow torch) and once the heat source is removed the fire will go out; repair of an open wood deck system will be simpler/cheaper than any similar repair to a concrete topping deck system; stains can be removed from wood by sanding the surface while stains in concrete may be more difficult to remove without the use of chemical cleaners or sandblasting.

Alternatively, a concrete walkway surface could be created using a concrete topping system similar to the existing east side of the widened trestle but will require a stiffer substructure to withstand emergency vehicle loads. The system would be about 27” deep and consist of a 3” concrete topping over a 6” nail laminated solid wood deck supported by 18” deep stringers in turn supported by the pile caps. We recommend the concrete topping be “soft cut” into 2’ squares and have contraction joints every 10’. While such a concrete surface would have certain advantages in terms of durability, it may be more complicated and expensive to get through the topping and nail-laminated deck to repair the substructure than an open wood deck system. While concrete does offer a canvas for alternative textures, colors and other surface treatments, when repairs are needed matching custom concrete surface treatments is difficult and often results in a patchwork appearance.

Given that costs of an open wood deck and a concrete topping are similar: between $150 to $170 per SF to construct, we recommend the open wood deck system.

Second, the pedestrian sidewalk between parking areas and the existing retail/restaurant buildings is presently a mix of AC and concrete pavement over wood deck. Because of the pedestrian loading on the sidewalk, upon further consideration, we believe that there are some advantages to the use of a light weight reinforced concrete paving treatment instead of the wood deck over wood sleeper system originally proposed. In addition, if concrete is utilized here it would best be achieved with a full 12-inch deep section sloped to drain toward the curb.

Recommendation: We recommend that the curb be changed to concrete and that a waterproof membrane be installed between the new concrete pavement and the supporting wood deck planks below. The sidewalk should be constructed with ADA compliant curb ramps much like any sidewalk elsewhere in the city. We estimate this lightweight reinforced concrete topping system would cost about $35 per SF (less than the previously proposed hardwood decking over sleepers system).

Attachment 6A City Emergency Response Plans

SUBJECT Utilization of units 771 and 778

PURPOSE Due to the size and weight of units 771 and 778, their operational utilization may be restricted in areas with poor access or weight limits. For standardization in operational deployment, the following guidelines have been established.

POLICY The duty battalion chief or incident commander has the discretion to deploy units 771/778 as needed**.**

Wharf Deployment

- ***ONLY 771*** meets the requirements to operate on the municipal wharf for emergency operations only. Refer to SOP #2-7.0 “Municipal Wharf Fire Operations”.

Strike Team Request

- Units 771 and 778 **do not** respond on strike team requests.

Mutual Aid Request

- Units 771 and 778 are to only accept a mutual aid request in which the location is safely accessible. If company officer is unsure of the safety regarding deployment at the incident, clarify with the IC.
- Paradise Park- Units 771 and 778 are not to respond into Paradise Park (exception- Per a SCZ Incident Commander request)

Cliff Rescue

- Cliff Rescue deployment of unit 771 or 778 will be at the discretion of the assigned officer(s) or Incident Commander.

Incident Deployment

- Operationally and in CAD, when unit 771 is *3114* it is an engine(with an aerial ladder), when unit 771 is 3171 it is a Truck.
- Officers/Engineers assigned to unit 3114, should be prepared to position for Truck Operations at any incident.

Training

- Training evolutions should be designed to limit reflex time for the truck to respond to emergency incidents.
- Acting Engineer may train on *3114* at his/her current program level (driving at Company Officer discretion). Assigned driver (classified spot) must have completed Acting Engineer Program.

Approved:

________________ _________________________

Jeff Trapp Date
Fire Chief

Santa Cruz City Fire Department
Standard Operating Procedure – SOP #2-7.0 Revised: 12/2/13

SUBJECT: Municipal Wharf Fire Operations

PURPOSE: To provide an initial response plan, information and tactical considerations for the effective control of fire and life safety.

POLICY: The municipal wharf represents a significant fire suppression challenge due to its construction characteristics and life safety concerns. The deployment of Fire Boat 3161 shall be included in the initial response. See SOP #

A. Company Operations

1. Kiosk staff at the main entrance will be instructed by the first-in company to restrict wharf access to emergency equipment only, until otherwise advised.

2. First-in company will proceed onto the wharf for size-up and this will include the determination - confirmation of above or below deck fire. If confirmed below deck fire of potential significance, the Incident Commander will request US Coast Guard and City of Monterey Fire firefighting vessels, harbor patrol and lifeguards.

3. Staging- Additional companies *will* stage at the entrance to the wharf. Restrict all wharf access if not being performed by Kiosk staff and request Police Department.

   Truck 3170 may **NOT** drive on the wharf at anytime. E3114/T3171 may drive on the wharf. **There is to be NO outrigger deployment on the wharf.**

4. Activated below deck sprinkler systems will greatly diminish the available above deck water system.

5. Apparatus travel speed shall be no greater than 15 miles per hour while operating on the wharf.

6. Hydrants utilized on the wharf will be those in a location that are between the wharf entrance and pumping engine. Parking apparatus beyond the fire area needs to be avoided(wharf structure failure-escape routes).

7. Apparatus spotting will be in a location that the apparatus is centered over a bent, placing axles in separate bays. Bents are located approximately 15 feet apart.

   a. For emergency operations, apparatus will spot a minimum of 30 feet away from other apparatus. This will assure adequate weight distribution in separate wharf bays. (No parallel parking.)

   b. For non-emergency operations, for two apparatus, 90 feet minimum separation, for three vehicles, 120 feet minimum separation.

8. Rescue and evacuation of occupants will normally be to exit off the wharf. City wharf crew personnel should be requested and retained for technical and mechanical expertise. (Wharf evacuation plan and call list in battalion chief vehicle.) The Police Department should be requested to control the intersection at the foot of the wharf.

9. Evacuation: The primary method of wharf evacuation will be to walk or drive off the wharf. Emergency method of evacuation would be by use of landings. All six landings may be commandered and utilized as evacuation points. In situations requiring this operation, the following agencies should be requested:

   a. U.S. Coast Guard (Monterey),
   b. Harbor Patrol
   c. City Marine Rescue team
   d. Private commercial marine vessels (party boats).
   e. City of Monterey Fire Boat
   f. State Parks Lifeguards
   g. Wharf Crew

   Southern Pacific and Big Trees Railroad should be notified to cease service to facilitate access and egress.

B. Above Deck/Structure Fires

1. Fires involving structures above the deck will follow conventional fire attack and control procedures. All buildings on the wharf are fully fire sprinklered. Primary system support to fire protection systems and hydrants is to supply the wharf entrance four-plex.

2. Fires that pose any threat or eminent spread to the deck, joists or below deck area, an additional fire company *will* be assigned to this area for surveillance and monitoring.

3. Deck access hatches are located(green dots) per the target hazard plan. Personnel assigned to below deck operations shall don life jackets, if breathing apparatus is not indicated.

4. Water supply feeding all sprinkler systems (above and below deck) and fire hydrants are from a 10" dead end main. This main changes to 6” at bent 147, and to 4” at bent 160. The FDC to augment all water supplies on the wharf is located at the foot of the wharf entrance. The fire hydrant located just prior to the FDC is off city mains. *Recommended not to use building specific FDC.

C. Below Deck Firefighting

1. First-in company shall attempt to locate and confirm the location and extent of the fire, in addition to activation status of below deck sprinkler systems.

2. The intensity and fire behavior prediction will be cause for activation of SOP #5-1.4 "Wharf Evacuation Plan".

3. Wharf tenants, Santa Cruz Police, and the parking kiosk will be directed to implement the wharf evacuation plan by the Fire Incident Commander, coordinated by SCPD(3103/3109).

4. Strategic objectives for below deck fire operations are to contain and extinguish the fire between the sprinkler systems. The FDC at the entrance to the wharf shall be supplied on all below deck fires.

5. Wharf decking may be opened for the purposes of inspection, deployment of hose streams, distributor nozzles and ventilation.

   Asphalt decking surface may be cut utilizing the rotary saw with the masonry composite blade. For extended operations, jackhammers will need to be utilized from wharf headquarters.

D. Marine Vessel Fire\Exposure

1. The rescue and retrieval of vessel occupants is the primary objective.

2. If occupants have abandoned the vessel, a flotation device should be thrown, and the occupants advised to swim away from the vessel. Marine Rescue and Harbor Patrol response will be immediately requested.

3. Appropriate exposure protection measures will be implemented if any portion of the wharf structure is threatened.

4. Vessels secured at landings will remain at that location, and firefighting operations implemented. (Prevent vessel from floating underneath wharf).

5. The primary extinguishing agent for marine vessel fires shall be foam. (Class B)

6. Department of Fish and Game, U.S. Coast Guard, and Harbor Patrol will be advised of any marine vessel fire.

E. Resources – Expanded Fire Operations

1. Confirmed structure response consists of: 4-E1, 1-TI, 1-DC, Santa Cruz Marine Rescue, Harbor Patrol, Command/Tactical frequencies. Truck 3170 shall **NOT** drive on the wharf at anytime. Truck 3170 is to stage at the entrance to the wharf under the direction of the Incident Commander. At the time of the alarm, if E3113 is at station 3, they **shall** respond in T3171.

2. Second alarm units per CAD is 3 E1, 2-D/C, 1-BS. 2nd alarm engines are staged at the Wharf entrance. Personnel with equipment are transported to the scene by a

SCMR member. One engine(1st or 2nd alarm) may be assigned to the ”systems” four-plex. Objective is to minimize the number of units on the wharf.

3. First arriving Marine Rescue person reports to the incident commander. Marine Rescue Intervention Crew(M-RIC) is staffed by Santa Cruz Marine Rescue Personnel for water safety and firefighter rescue. They will coordinate with other resources specific to water rescue. “M-RIC” will be assigned to the fire supervisor assigned as IRIC.

4. Operations for T3170 for the Municipal Wharf shall consist of the following:
    - T3170 shall stage at the entrance to the wharf.
    - Personnel will be transported to the emergency scene by the Santa Cruz Marine Rescue Personnel or other transportation necessary as determined by the Incident Commander.
    - In the event that a 35’ ladder or other equipment from T3170 is needed at the emergency scene, the following vehicles maybe considered for transport: Marine rescue, Wharf Maintenance or fire department vehicles. Anyone of which must be approved by the Incident Commander.

F. Municipal Wharf - Information

1. Resources:
    - United States Coast Guard (Monterey), 1.5 hr. response
    - Harbor Patrol
    - Marine Rescue Team - water rescue
    - Helispot - 30 ton weight capability
    - City of Monterey Fire Boat

2. Wharf landings (6) from entrance to end are known as:
    - Kayak landing
    - Santa Cruz Boat Rental
    - Public Landing #1 (stairwell may be raised during winter months)
    - City Landing
    - Stagnaro Landing
    - Public Landing #2 (stairwell may be raised during winter months)
      (All landings <u>should</u> have 3476 lock)

3. Wharf - general information
    - Wharf hours, open - 0500 hrs./close 0200 hrs.
    - When closed, gate arms are up, main gates unlocked
    - All buildings are sprinklered
    - 22 under deck sprinkler systems, 100 ft ± apart on a continuous line
    - Parallel to the water main valves for sprinklers: up to bent <u>38</u> under diamond plates on west side of wharf; others past <u>38</u> on east side under diamond plate
    - 10 hatches (near walkways on westside near businesses). Extra hatch in bathroom at end of wharf (ladder only)

- Deck lids access below deck sprinkler valves, no fire department connections
- 10" water main to 4" dead end
- 125 psi static water main
- Local winds, 1100 hrs. - 1800 hrs.; SW15-20 mph, opposite at night

4. Wharf construction terminology is defined as follows:

Piling: Vertical post from ocean floor
Cap: Horizontal beam, perpendicular to wharf length, which spans and connects pilings.
Joists: Horizontal members, on edge, running parallel to wharf length. Rests on caps and supports decking.
Decking: Top most wooden planks. Rests on joists, covered with asphalt or concrete.
Bent: Main structural component comprised of pilings and caps. Perpendicular to length of wharf.
Bay: Those areas of structure between bents.

- 2700' length; 250' widest point
- Mean distance deck to ocean, 20 ft.
- Available wharf construction workers, 6
- Deck spikes 7" x 5/16"
- Average distance between bents, 15 ft.
- Wood preservatives, creosote and ACZA

Approved:

______________________________ _________________________
Jeff Trapp Date
Fire Chief

Santa Cruz City Fire Department
Standard Operating Procedure – SOP #2-7.1 12/2/13

SUBJECT: Truck 3170 Emergency Medical Response Procedures to the Municipal Wharf.

PURPOSE: To establish guidelines for Truck 3170 responses to the wharf

POLICY : Under direction of the on duty Battalion Chief, on duty fire department personnel assigned to Truck 3170 will use the following criteria when responding to the wharf for medical emergencies.

1. **If dispatched to a medical emergency and another type one engine becomes available, depending on location of that type one engine, a unit exchange through Net Com may be administered. Communication between both Captains shall be done to see if the unit exchange is feasible.**

2. If there are no type one engines available or a unit exchange is not feasible with an available type one engine Truck 3170 shall respond, with R3161 from Station 1, to the head of the wharf. Staging of Truck 3170 shall be at Cowell's parking lot or another suitable location as deemed appropriate by the Truck Captain.

3. The following additional options shall be considered by the Truck Captain upon arriving at the entrance to the wharf:

   A. Request 3103 code 3 to the scene to assist with transporting personnel and equipment to the emergency scene.
   B. Request SCPD code 3 to assist with transporting personnel and equipment to the emergency scene.
   C. Request the on duty lifeguard vehicle to respond to the entrance to the wharf for transportation of fire personnel and equipment to the emergency location. Lifeguard 3108 and T3170 may need to plan the course of action for transportation of personnel and equipment to the emergency scene.
   D. Truck 3170 Captain may elect to bring another Fire Department Vehicle to the scene to facilitate equipment and personnel transport to the emergency scene – R3171

4. These Procedures are not to preclude other options that may be available to the Truck Captain to assist with the emergency incident.

Approved:

Jeff Trapp
Fire Chief

Date

Santa Cruz City Fire Department
Marine Safety Division
Standard Operating Procedure – SOP #3-1.0 Revised 2/22/12

SUBJECT: Lifeguard Medical Aid Response in Beach/Wharf/Boardwalk Area

PURPOSE: To establish procedures for medical responses in the beach area.

POLICY: Under direction of the Battalion Chief, beach Captain or first arriving fire officer, lifeguard personnel will respond to incidents in the beach area using the following criteria:

1. Response Zones:
    - A. Wharf and entrance parking area (south of Beach Street)
    - B. The Boardwalk
    - C. Third Street parking lot
    - D. Main Beach and adjacent sidewalks
    - E. Cowell's Beach and parking area
    - F. Other areas as requested by the Battalion Chief

2. Hours and Staffing:

   Anytime two lifeguard/EMT's with current certifications are staffing a lifeguard vehicle.

3. Types of Calls:

   Medical Aid Calls

4. Operations:
    - A. When paged by Netcom, a fire department suppression unit will respond to medical calls concurrently with the lifeguard unit. Depending on the call type and the current ocean conditions and staffing Lifeguards may elect not to respond out of the beach area.
    - B. The first arriving fire department officer will become the scene manager.
    - C. The first fire department unit to arrive on the scene will assess the situation and cancel the second responding unit, if appropriate.
    - D. A canceled unit shall become immediately available and return to its previous assignment and/or quarters.
    - E. The fire officer will be responsible for completing the required run report information unless they are canceled.

Approved:

________________________ ________________________

Jeff Trapp Date
Fire Chief

Santa Cruz Fire Department
Standard Operating Guidelines – SOG #5 New 5/10/13

SUBJECT: Fire Boat Operations and Deployment, Type V Fire Boat (NFPA 1925)

PURPOSE: To provide guidelines and expectations for the response and operation of the Fire Boat.

POLICY: The Fire Boat is a specialized piece of firefighting equipment. It demands specialized training and ongoing familiarization for the entire department.

A. Fire Boat Operations

1. The Fire Boat will require a minimum of 3 personnel including a department certified pilot to deploy and operate.
2. The on duty Battalion Chief will assign an engine company to respond to the boat for operation. The decision on who will respond depends on the mission and who is certified, similar to how the rescue swimmers are deployed.
3. The Fire Boat is limited to daylight operations at this time.
4. The response area is limited to the Santa Cruz side of the bay and will be dependent upon ocean conditions.
5. Response types will be for wharf fires, boat fires and hazardous material response. All other missions will be evaluated with the on duty Battalion Chief and the pilot at the time of the incident.
6. The Fire Boat may be used for water rescues as a last resort. Utilize the PWC's and harbor patrol first.
7. The primary mission of this boat is water delivery to fires on the wharf and fires involving vessels at the harbor or in the bay.
8. The company assigned will need to bring their PPE and breathing apparatus with them to the boat. Hose, nozzles and hand tools are stowed on the boat.
9. Communications will be established the same as we do for multi-unit responses. Net com will assign a command and tactical channel. The pilot will also maintain marine communications via marine channel 16.
10. All crewmembers will don the inflatable flotation devices during emergency operations.

B. Crew Assignments

1. The Fire Captain will be in charge of communications and tactical decisions. The Captain will also act as a spotter for fire stream placement and effectiveness.
2. The Pilot will determine the capabilities of the mission and operations of the boat. The Pilot will have the authority to abort the mission if conditions warrant it.
3. The Firefighter will secure the dock lines and set up equipment for the assigned operations. This includes setting up hose lines and operating the turret.
4. All crew members will act as spotters and assist as directed by the pilot.

C. Specifications

1. Length 34'6", draft 44", beam 11'9", displacement 19,000lbs.
2. 2 Caterpillar Model 3208 Marine Diesel engines, horsepower 203 at (2800 RPM)
3. Maximum speed 20.4 kts. at 2800 RPM, cruising speed 16.5 kts. At (2400 RPM)
4. Fuel capacity 200 gallons, range 190 miles
5. Detroit Diesel powers a Waterous CLNT – 500 pump, 500 GPM's @ 210 PSI / 2800 RPM's
6. (1) 500 GPM deck gun variable pattern
7. (1) 2.5" gated wye to 2 1.5 " gated outlets
8. Built in foam eduction system for class B foam operations.
9. BLS / O2 medical equipment

D. Basic Firefighting Tools and Equipment

1. 24” bolt cutters
2. Halligan
3. 6# flat head axe
4. 100’ rope
5. 100’ 2.5” hose
6. 300’ 1.75” attack hose
7. (2) pocket spanners
8. (2) hose straps
9. 2.5” double female adapter
10. 2.5” double male adapter
11. 5’ pry bar
12. 30# CO2 extinguisher
13. 2A10BC dry chem extinguisher
14. (2) 60 gpm adjustable nozzles
15. (1) 250gpm fog nozzle
16. (1) Pike pole

7. WHARF STRUCTURE SUPPORTING BUILDINGS

7.1 Summary

The Wharf structure has adequate structural capacity to support one- and two-story buildings, including those identified in companion Master Plan. Future buildings on the Wharf, should follow the recommendations in this section to avoid loads from being improperly located on the Wharf structure. When buildings are replaced, the supporting wharf structure should be thoroughly inspected and all deteriorated members (decking, stringers, caps and piles) replaced. The Wharf structure beneath the building occupied by Miramar Restaurant was examined for consideration of replacement during this report.

7.2 Introduction

The Santa Cruz Wharf currently supports multiple buildings, including a life guard building and many commercial spaces. Part of the Master Plan is to evaluate the construction of new buildings or alterations to the existing ones. Preliminary calculations were performed (Mesiti-Miller Engineers) for new one- and two-story vertical building loads. These loads were then analyzed for the capacity of the Wharf substructure to support these loads.

7.2.1 Scope

- Evaluate structural integrity of substrate and identify any weak or vulnerable areas.
- Provide longevity estimates for existing substrate.
- Provide general construction recommendations for structural support of new single and two-story buildings at various points along the Wharf where the water depth varies.

7.2.2 Description of Structure

The Santa Cruz Wharf currently supports numerous buildings; most of them are along the west edge of the Wharf, starting at bent 70 to end of the Wharf. All of the buildings are either one or two stories.

7.2.3 Methodology

The condition assessment of the Wharf substructure is described in Sections 1 and 2. Vertical load calculations were performed (Mesiti-Miller Engineers) for new one- and two-story buildings, these loads are analyzed for the capacity of the Wharf substructure to support these loads.

7.3 Condition Assessment

See Sections 1 and 2 for discussion of the Wharf structural condition. In general, no deficiencies were observed that would prevent new buildings being constructed. The most deterioration of members and missing piles (with replacement A-frames) were found under the existing buildings, in particular the Miramar Restaurant.

7.4 Analysis

Loads for one- and two-story buildings, were analyzed that include floor live load, bearing wall line loads and interior column point loads and are provided in Attachment 7A. Using these loads the substructure is analyzed to determine whether the existing structure can support new building loads. These calculations can be found in Attachment 2C.

- Building columns are assumed to be on a 20 ft. by 20 ft. grid.

Attachment 7A Building Loads on Wharf

MESITI-MILLER ENGINEERING, INC.
224 Walnut Avenue, Ste. B
Santa Cruz, California 95060
(831) 426-3186 FAX (831) 426-6607
www.m-me.com

JOB 13116 SANTA CRUZ WHARF
SHEET NO. B1 OF 4
CALCULATED BY DH DATE 11-13-13
CHECKED BY DATE
SCALE

WHARF STRUCTURE SUPPORTING BUILDINGS

SUMMARY OF BUILDING LOADS

i) PROPOSED ONE STORY BUILDING:
ASSUMED 20 ft × 20 ft COLUMN GRID
HEIGHT = 14 ft

a. BEARING WALL UNIFORM LINE LOAD -

$W_{DL} = 400\ \#/ft.$
$W_{LL} = 200\ \#/ft.$
$W_{TOTAL} = 600\ \#/ft.$

b. CONCENTRATED INTERIOR COLUMN LOAD -

$P_{DL} = 6{,}500\ \#$
$P_{LL} = 6{,}400\ \#$
$P_{TOTAL} = 12{,}900\ \#$

---

ii) PROPOSED TWO STORY BUILDING:
ASSUMED 20 ft × 20 ft COLUMN GRID
HEIGHT = 24', CONCRETE TOPPING ON 2ND FLOOR

a. BEARING WALL UNIFORM LINE LOAD -

$W_{DL} = 800\ \#/ft.$
$W_{LL} = 1200\ \#/ft.$
$W_{TOTAL} = 2{,}000\ \#/ft.$

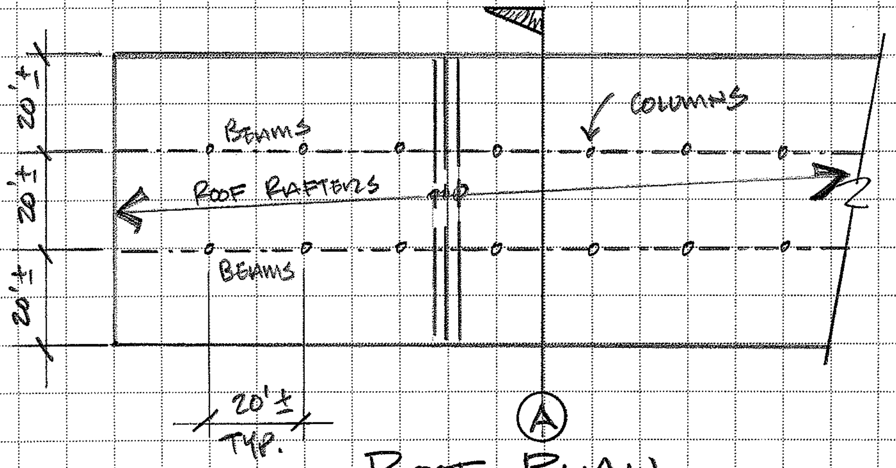
b. CONCENTRATED INTERIOR COLUMN LOAD -

$P_{DL} = 18{,}700\ \#$
$P_{LL} = 31{,}400\ \#$
$P_{TOTAL} = 50{,}100\ \#$

JOB 13116 S.C. WHARF
SHEET NO. B2 OF 4
CALCULATED BY DH DATE 11-1-13
CHECKED BY DATE
SCALE

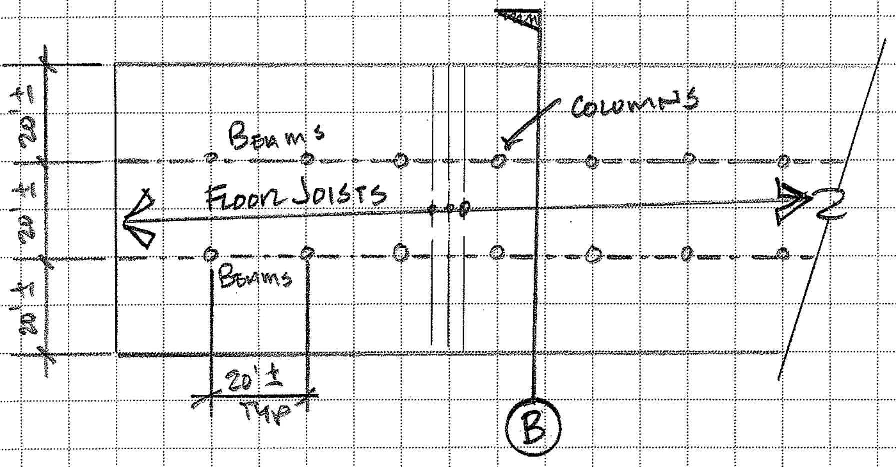
TYPICAL BUILDING CONFIGURATIONS

i) ONE STORY BUILDING -



ROOF PLAN

ii) TWO STORY BUILDING -

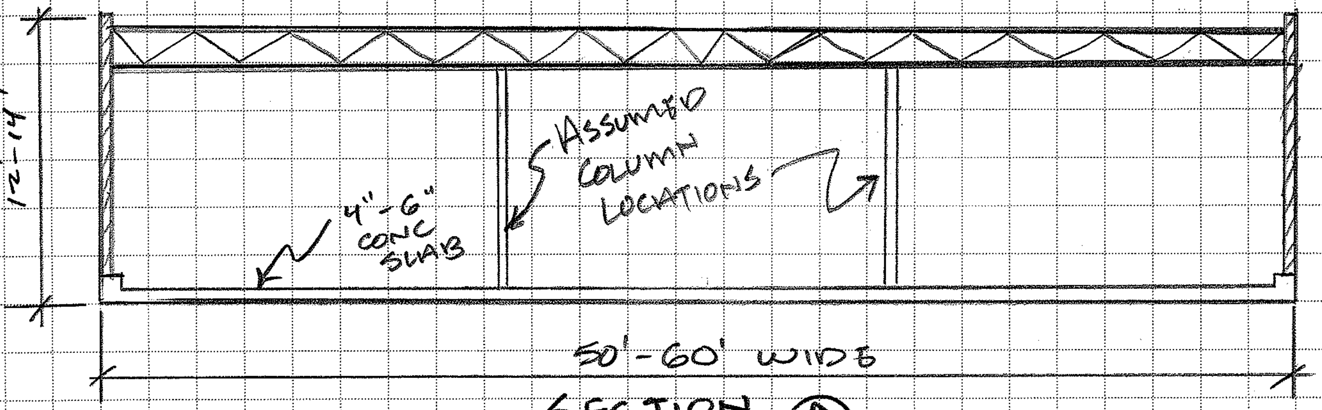


ROOF/FLOOR PLAN

JOB 13116 S.C. WHARF
SHEET NO. B3 OF 4
CALCULATED BY DH DATE 11-1-13
CHECKED BY DATE
SCALE

TYPICAL BUILDING LOADS

i) ONE STORY BUILDING



SECTION Ⓐ

BUILDING WEIGHTS -

ROOF = 15 psf
EXTERIOR WALL = 12 psf
CONC CURB (8"x12") = 100 #/ft.
FLOOR SLAB = 75 psf

LOADS -

ROOF LL = 20 psf
FLOOR LL = 100 psf
FLOOR LL REDUCTION = $\left(0.25 + \frac{15}{\sqrt{4 \times 400}}\right) = 0.625 \times W_{LL}$

LINE LOADS -

$W_{DL} = \overset{\text{ROOF}}{(15)\left(\frac{20'}{2}\right)} + \overset{\text{WALL}}{(12)(14')} + \overset{\text{CURB}}{100^{\#}} = 418 \ \#/\text{ft.}$

$W_{LL} = (20)\left(\frac{20}{2}\right) = 200 \ \#/\text{ft.}$

POINT LOADS - TRIB. AREA = 20' x 20' = 400 ft²

$P_{DL} = \overset{\text{ROOF}}{(15)(400)} + \overset{\text{COLUMN}}{(15)(12')} + \overset{\text{FLOOR SLAB}}{(2')(2')(75)} = 6{,}480^{\#}$

$P_{LL} = (16)(400) = 6{,}400^{\#}$

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(831) 426-3186 FAX (831) 426-6607
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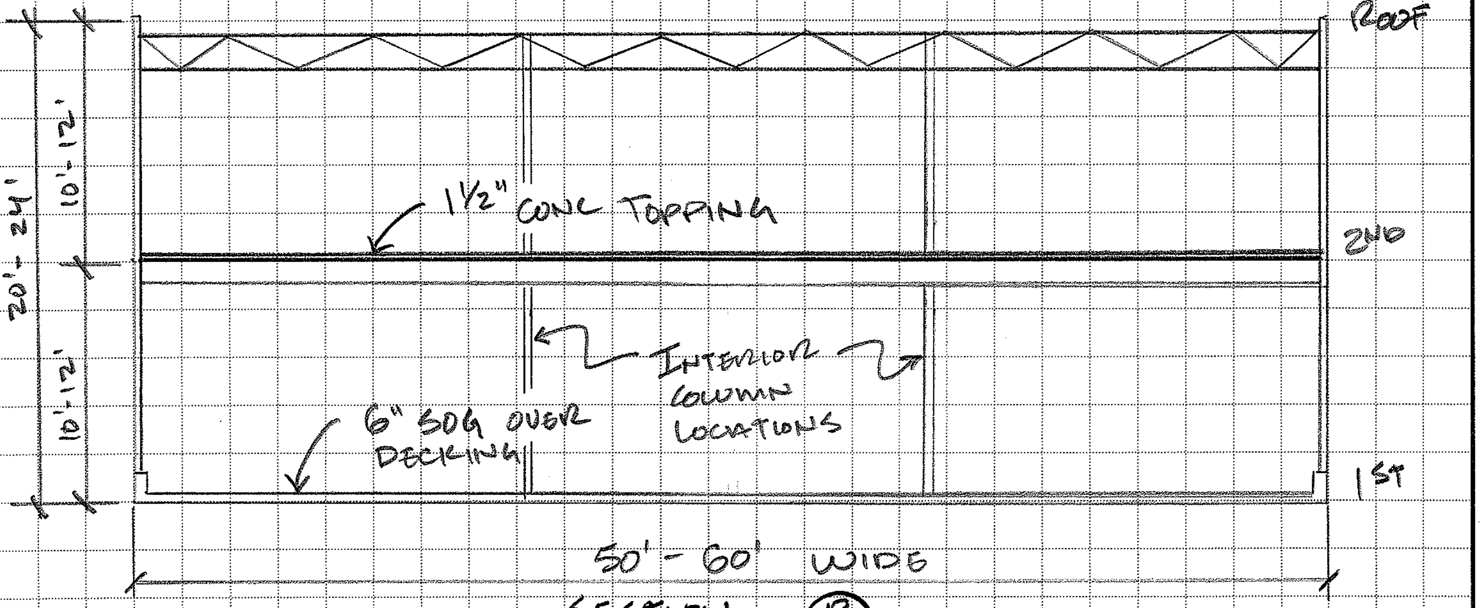
JOB 13116 S.C. WHARF
SHEET NO. B4 OF 4
CALCULATED BY DH DATE 11-7-13
CHECKED BY ______ DATE ______
SCALE ______

11) TWO STORY BUILDING



SECTION (B)

BUILDING WEIGHTS

	DL	LL	REDUCED LL (400ft²)
ROOF	= 15 psf	20 psf	16 psf
2ND FLOOR	= 30 psf	100 psf	62.5 psf
1ST FLOOR	= 75 psf	100 psf	

LINE LOADS

$$W_{DL} = (15)\overset{ROOF}{\left(\frac{20'}{2}\right)} + (30)\overset{FLOOR}{\left(\frac{20'}{2}\right)} + \overset{WALL}{(12)(24')} + \overset{CURB}{100} = 838\ \#/ft.$$

$$W_{LL} = (20)\overset{ROOF}{\left(\frac{20'}{2}\right)} + (100)\overset{FLOOR}{\left(\frac{20'}{2}\right)} = 1{,}200\ \#/ft.$$

POINT LOAD TRIB AREA = 20' x 20' = 400 ft²

$$P_{DL} = \overset{ROOF}{(15)(400)} + \overset{FLOOR}{(30)(400)} + \overset{COLUMN}{(15)(24')} + \overset{FLOOR\ SLAB}{(2')(2')(75)} = 18{,}660\ \#$$

$$P_{LL} = \overset{REDUCED\ ROOF}{(16)(400)} + \overset{REDUCED\ FLOOR}{(62.5)(400)} = 31{,}400\ \#$$

$$P_{TOTAL} = 18.7 + 31.4 = 50.1\ ^{k}$$

Attachment 10A Figures

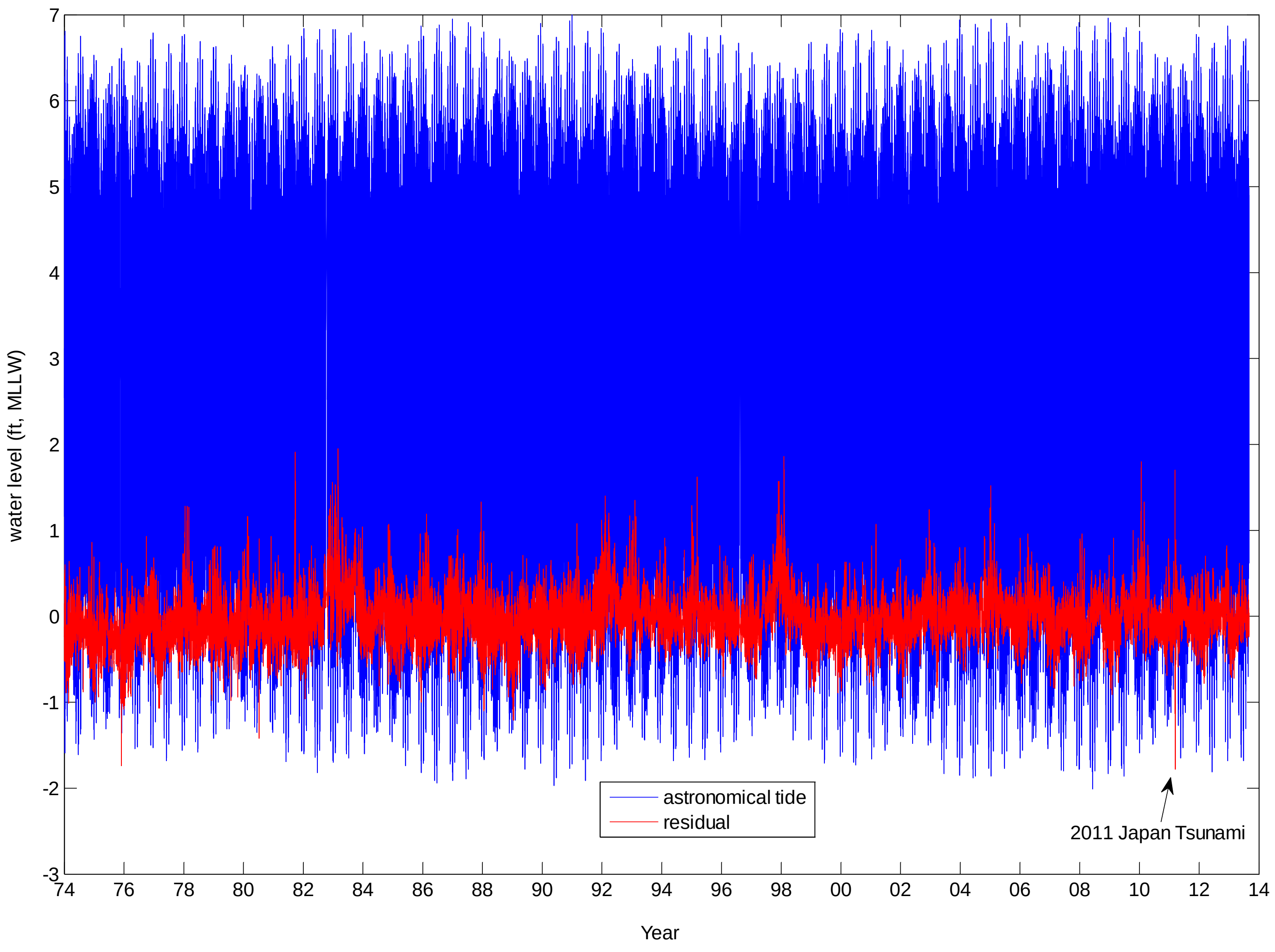


Figure 10-11: Time Series Plot of Astronomical Tides and Residual Tides for the NOAA Monterey Tide Gauge

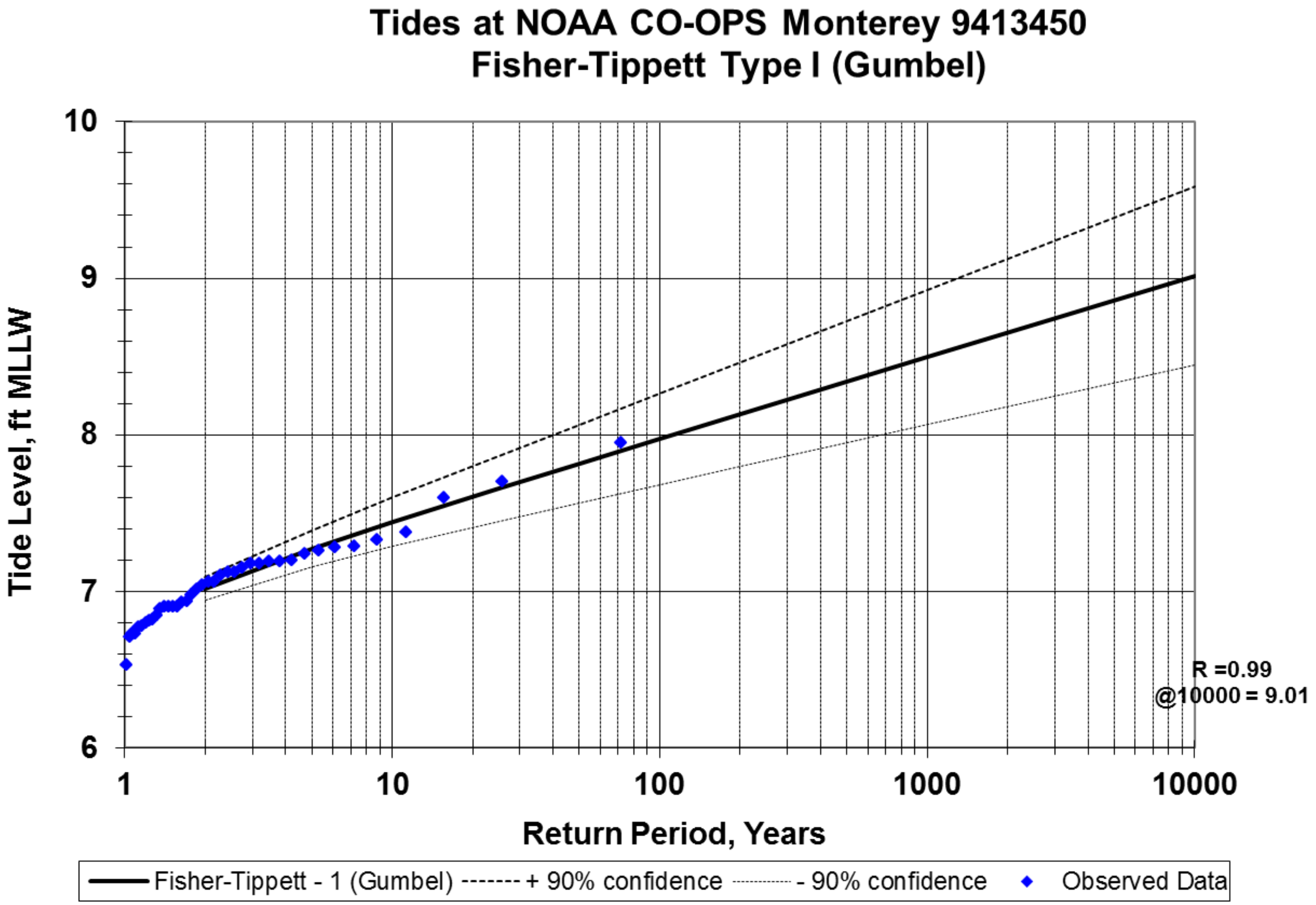


Figure 10-12: Extreme Tides Analysis for the NOAA Monterey Tide Gauge

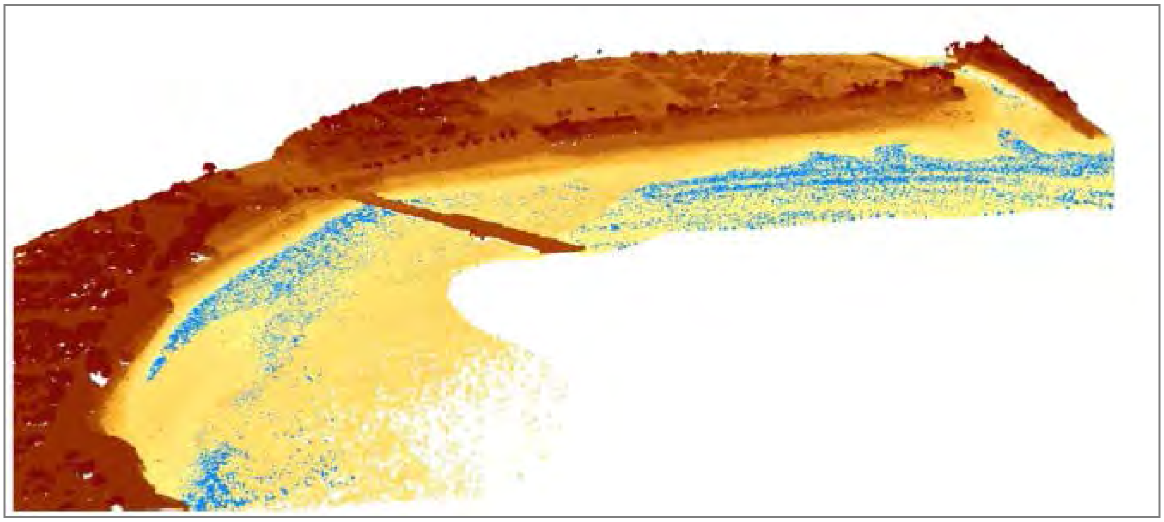


Figure 10-13: Main Beach and the Wharf at Mean High Tide in 2008 Condition (Extracted from Griggs and Haddad, 2011)

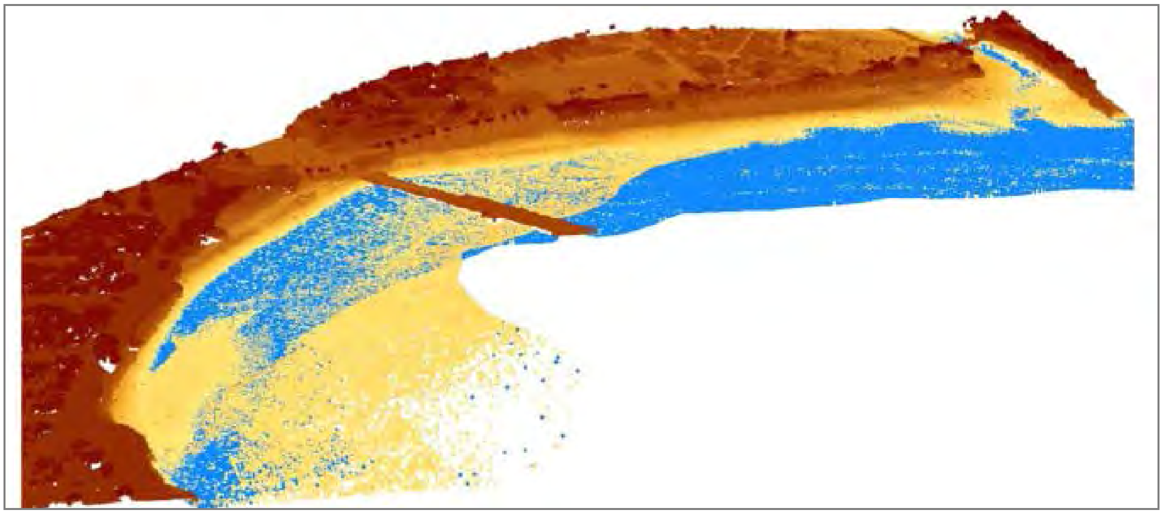


Figure 10-14: Main Beach and the Wharf at Mean High Tide with a 1-ft SLR Scenario (Extracted from Griggs and Haddad, 2011)

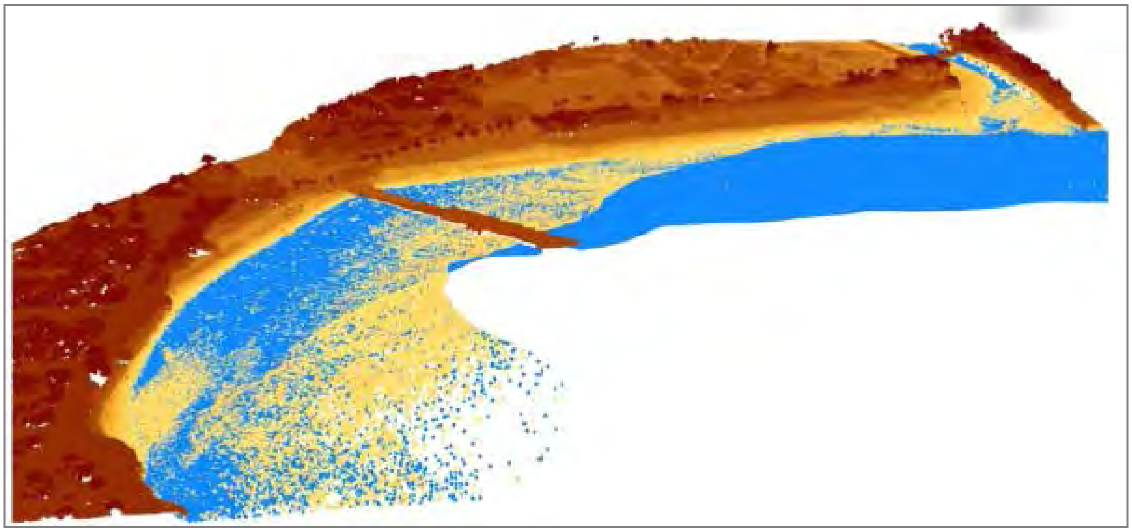


Figure 10-15: Main Beach and the Wharf at Mean High Tide with a 2-ft SLR Scenario (Extracted from Griggs and Haddad, 2011)

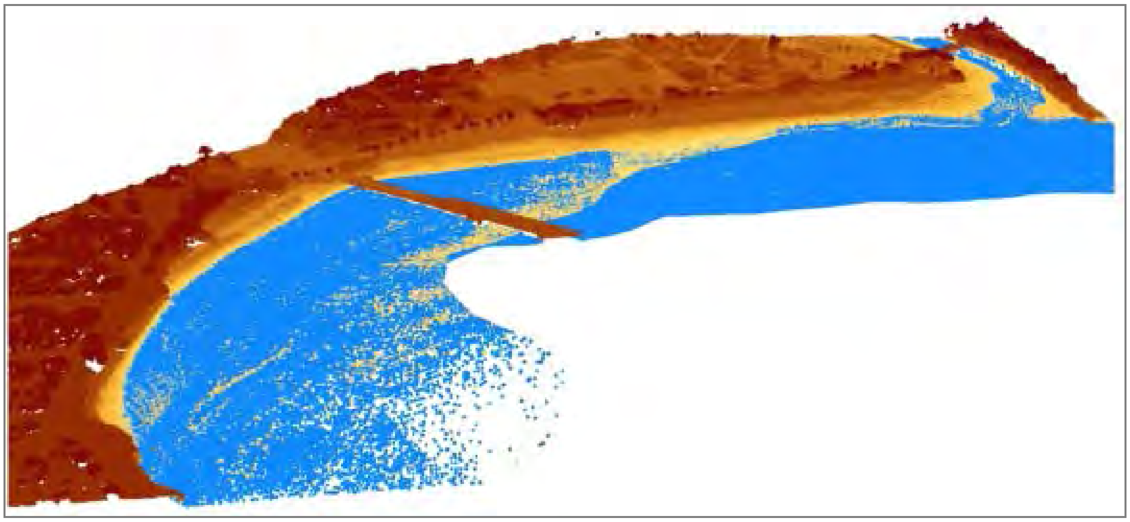


Figure 10-16: Main Beach and the Wharf at Mean High Tide with a 3-ft SLR Scenario (Extracted from Griggs and Haddad, 2011)

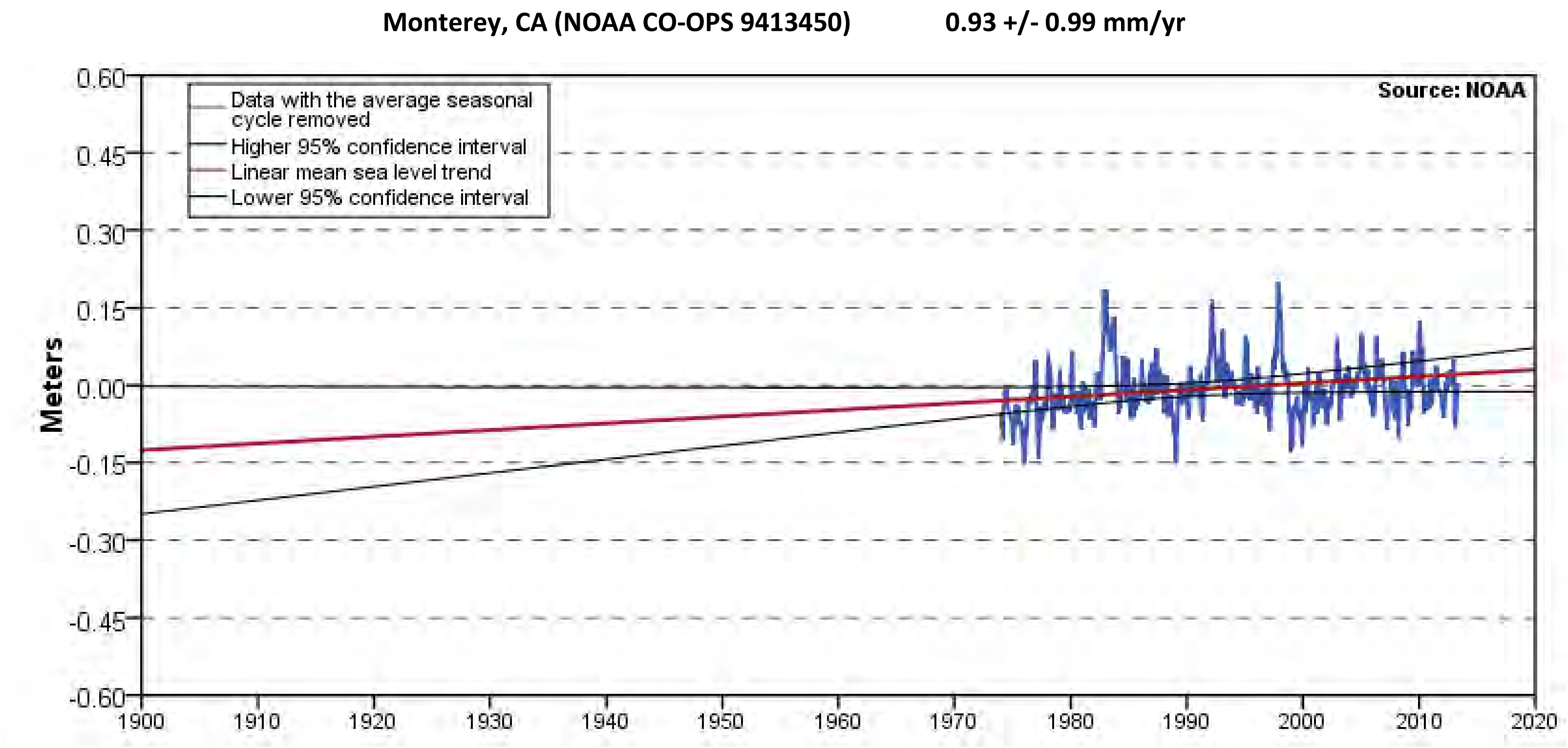


Figure 10-17: Mean Sea Level Trend of NOAA CO-OPS Tide Gauge at Monterey, CA from 1973 to 2012

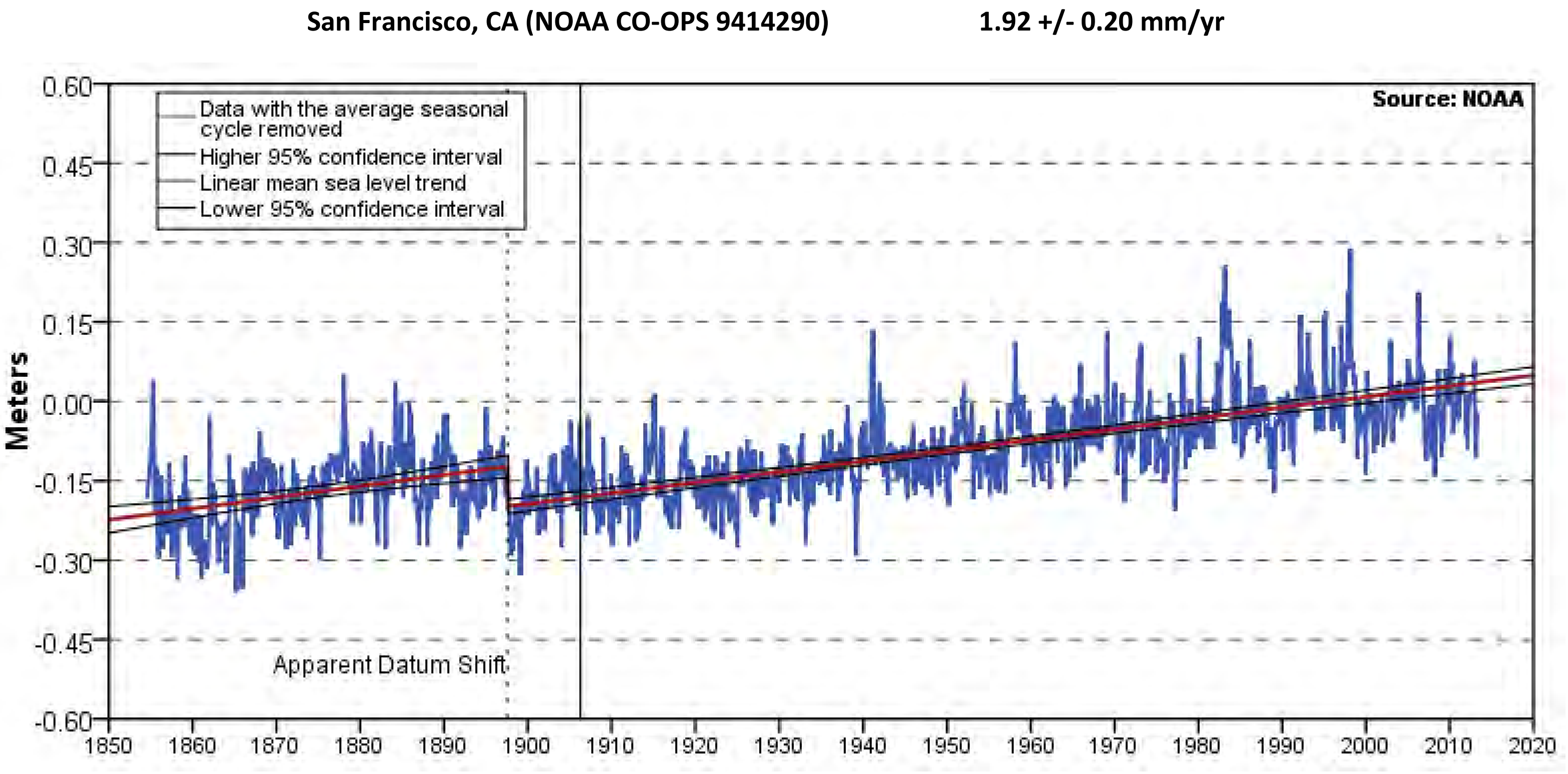
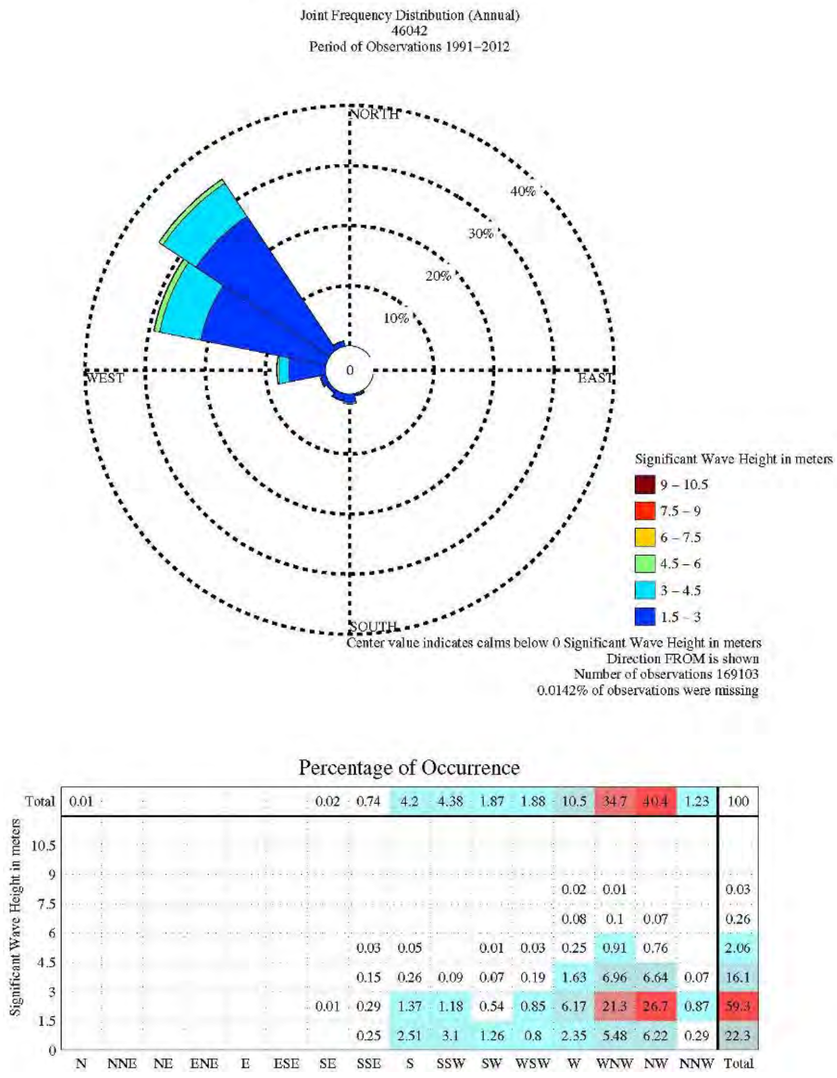


Figure 10-18: Mean Sea Level Trend of NOAA CO-OPS Tide Gauge at San Francisco, CA from 1897 to 2012



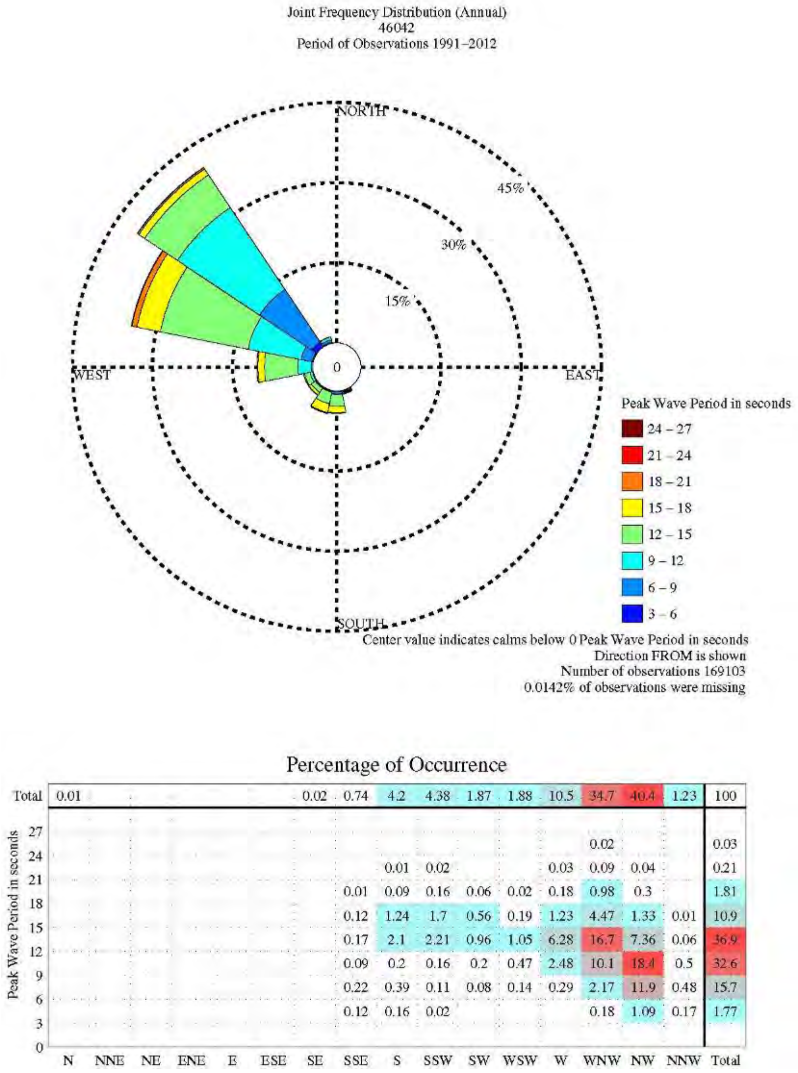
Joint Frequency Distribution (Annual)
46042
Period of Observations 1991–2012

Center value indicates calms below 0 Significant Wave Height in meters
Direction FROM is shown
Number of observations 169103
0.0142% of observations were missing

Percentage of Occurrence

	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	0.01						0.02	0.74	4.2	4.38	1.87	1.88	10.5	34.7	40.4	1.23	100
													0.02	0.01			0.03
													0.08	0.1	0.07		0.26
								0.03	0.05		0.01	0.03	0.25	0.91	0.76		2.06
								0.15	0.26	0.09	0.07	0.19	1.63	6.96	6.64	0.07	16.1
							0.01	0.29	1.37	1.18	0.54	0.85	6.17	21.3	26.7	0.87	59.3
								0.25	2.51	3.1	1.26	0.8	2.35	5.48	6.22	0.29	22.3

Figure 10-19: Annual Wave Height Rose at the Buoy 46042



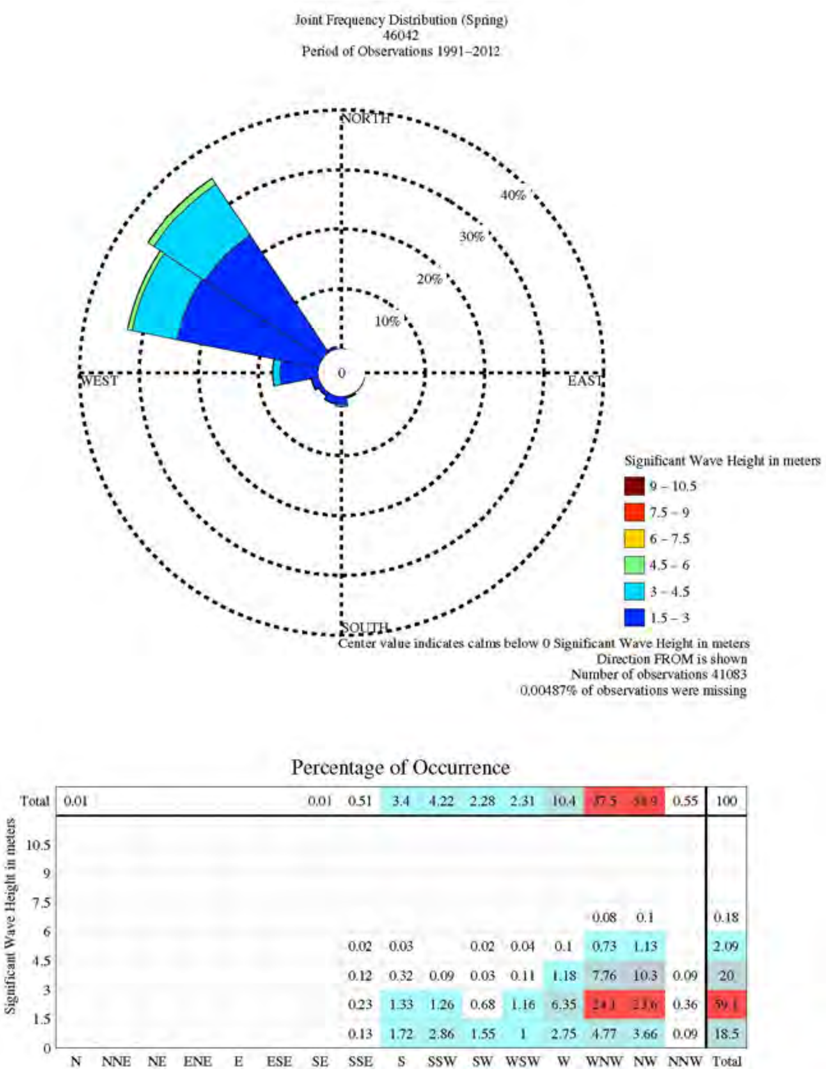
Joint Frequency Distribution (Annual)
46042
Period of Observations 1991–2012

Center value indicates calms below 0 Peak Wave Period in seconds
Direction FROM is shown
Number of observations 169103
0.0142% of observations were missing

Percentage of Occurrence

	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	0.01						0.02	0.74	4.2	4.38	1.87	1.88	10.5	34.7	40.4	1.23	100
														0.02			0.03
									0.01	0.02			0.03	0.09	0.04		0.21
								0.01	0.09	0.16	0.06	0.02	0.18	0.98	0.3		1.81
								0.12	1.24	1.7	0.56	0.19	1.23	4.47	1.33	0.01	10.9
								0.17	2.1	2.21	0.96	1.05	6.28	16.7	7.36	0.06	36.9
								0.09	0.2	0.16	0.2	0.47	2.48	10.1	18.4	0.5	32.6
								0.22	0.39	0.11	0.08	0.14	0.29	2.17	11.9	0.48	15.7
								0.12	0.16	0.02				0.18	1.09	0.17	1.77

Figure 10-20: Annual Wave Period Rose at the Buoy 46042



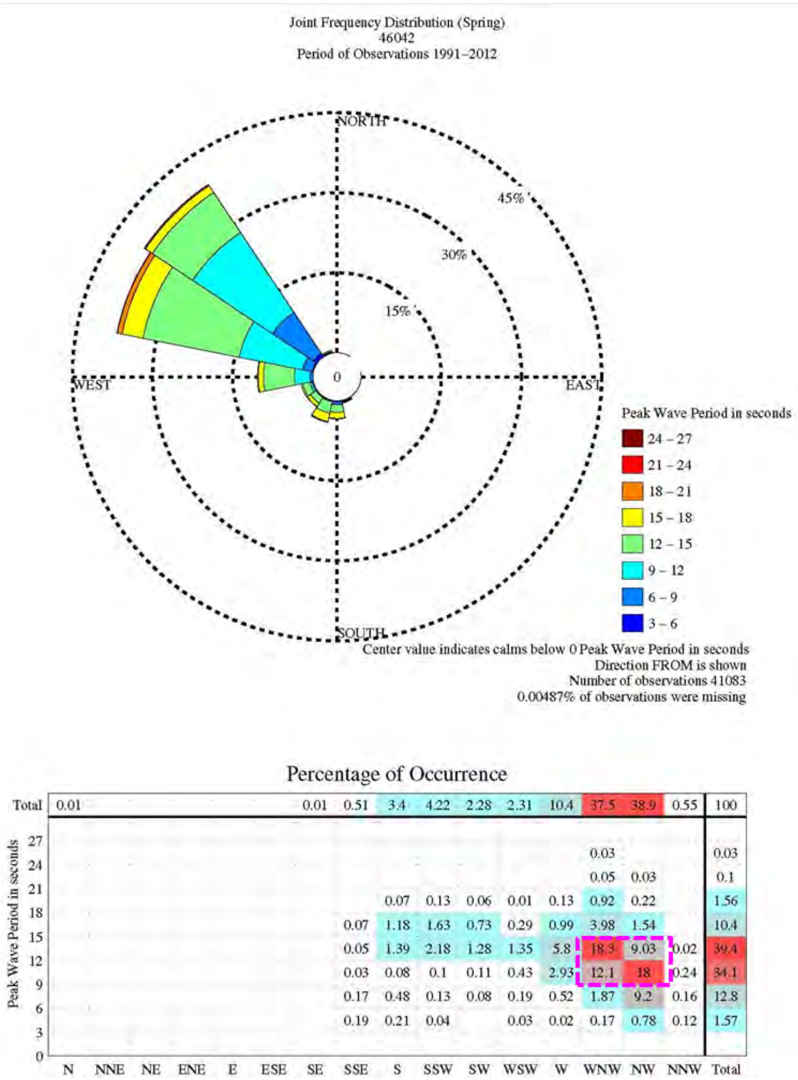
Joint Frequency Distribution (Spring)
46042
Period of Observations 1991–2012

Center value indicates calms below 0 Significant Wave Height in meters
Direction FROM is shown
Number of observations 41083
0.00487% of observations were missing

Percentage of Occurrence

	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	0.01						0.01	0.51	3.4	4.22	2.28	2.31	10.4	37.5	38.9	0.55	100
														0.08	0.1		0.18
								0.02	0.03		0.02	0.04	0.1	0.73	1.13		2.09
								0.12	0.32	0.09	0.03	0.11	1.18	7.76	10.3	0.09	20
								0.23	1.33	1.26	0.68	1.16	6.35	24.1	23.6	0.36	59.1
								0.13	1.72	2.86	1.55	1	2.75	4.77	3.66	0.09	18.5

Figure 10-21: Spring Wave Height Rose at the Buoy 46042



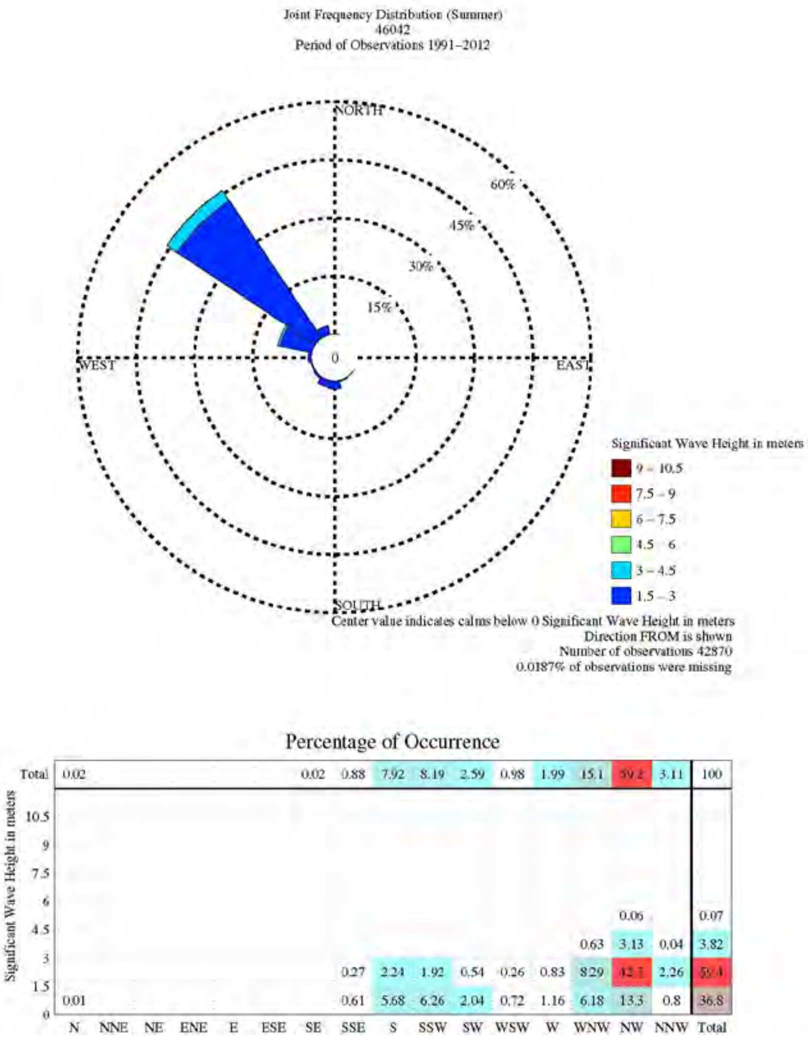
Joint Frequency Distribution (Spring)
46042
Period of Observations 1991–2012

Center value indicates calms below 0 Peak Wave Period in seconds
Direction FROM is shown
Number of observations 41083
0.00487% of observations were missing

Percentage of Occurrence

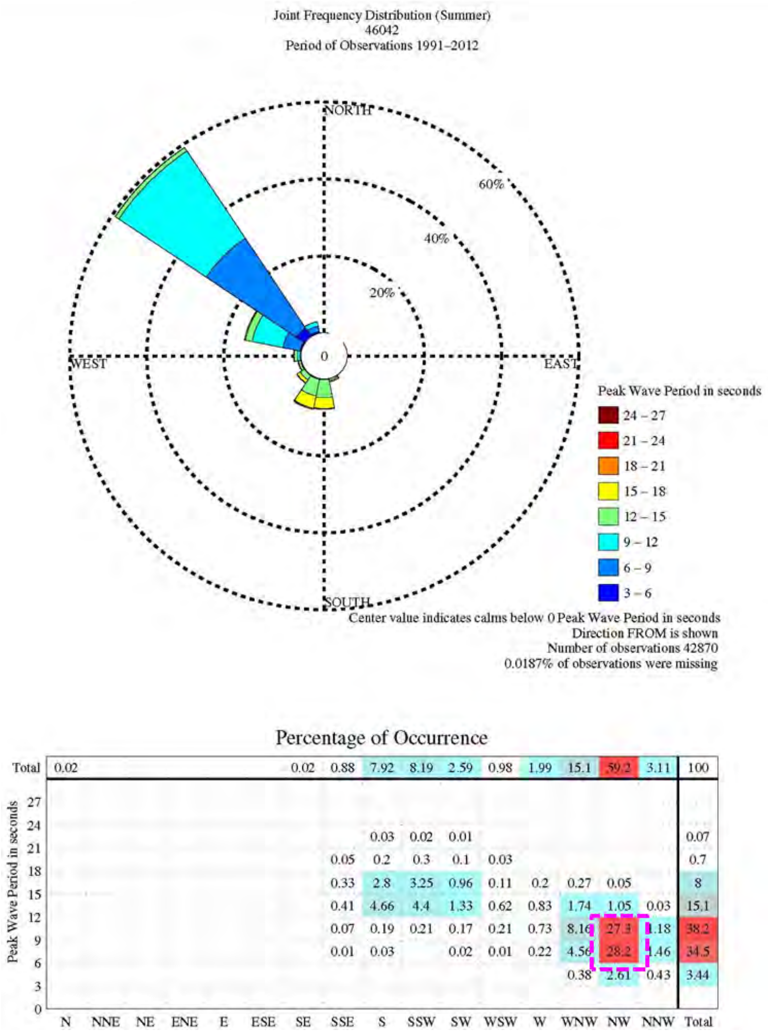
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	0.01						0.01	0.51	3.4	4.22	2.28	2.31	10.4	37.5	38.9	0.55	100
														0.03			0.03
														0.05	0.03		0.1
									0.07	0.13	0.06	0.01	0.13	0.92	0.22		1.56
								0.07	1.18	1.63	0.73	0.29	0.99	3.98	1.54		10.4
								0.05	1.39	2.18	1.28	1.35	5.8	18.3	9.03	0.02	39.4
								0.03	0.08	0.1	0.11	0.43	2.93	12.1	18	0.24	34.1
								0.17	0.48	0.13	0.08	0.19	0.52	1.87	9.2	0.16	12.8
								0.19	0.21	0.04		0.03	0.02	0.17	0.78	0.12	1.57

Figure 10-22: Spring Wave Period Rose at the Buoy 46042



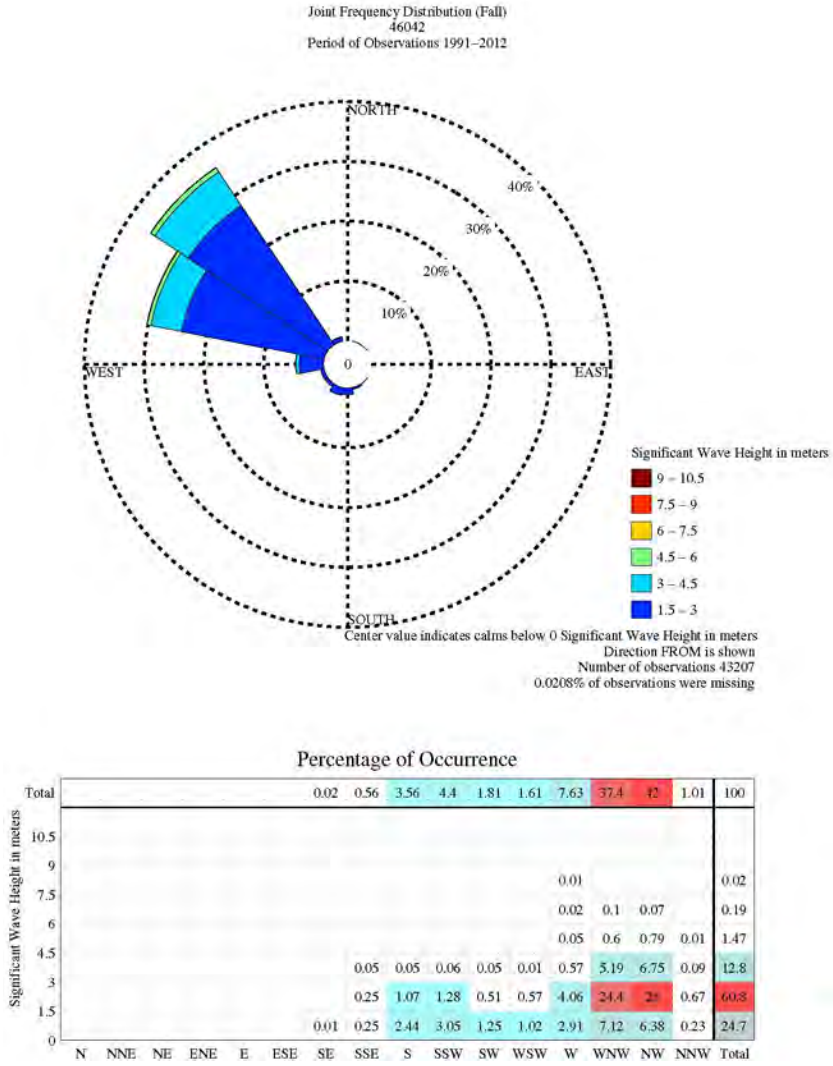
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	0.02						0.02	0.88	7.92	8.19	2.59	0.98	1.99	15.1	59.2	3.11	100
4.5–6															0.06		0.07
3–4.5														0.63	3.13	0.04	3.82
1.5–3								0.27	2.24	1.92	0.54	0.26	0.83	8.29	42.1	2.26	59.4
0–1.5	0.01							0.61	5.68	6.26	2.04	0.72	1.16	6.18	13.3	0.8	36.8

Figure 10-23: Summer Wave Height Rose at the Buoy 46042



	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	0.02						0.02	0.88	7.92	8.19	2.59	0.98	1.99	15.1	59.2	3.11	100
21–24									0.03	0.02	0.01						0.07
18–21								0.05	0.2	0.3	0.1	0.03					0.7
15–18								0.33	2.8	3.25	0.96	0.11	0.2	0.27	0.05		8
12–15								0.41	4.66	4.4	1.33	0.62	0.83	1.74	1.05	0.03	15.1
9–12								0.07	0.19	0.21	0.17	0.21	0.73	8.16	27.3	1.18	38.2
6–9								0.01	0.03		0.02	0.01	0.22	4.56	28.2	1.46	34.5
3–6														0.38	2.61	0.43	3.44

Figure 10-24: Summer Wave Period Rose at the Buoy 46042



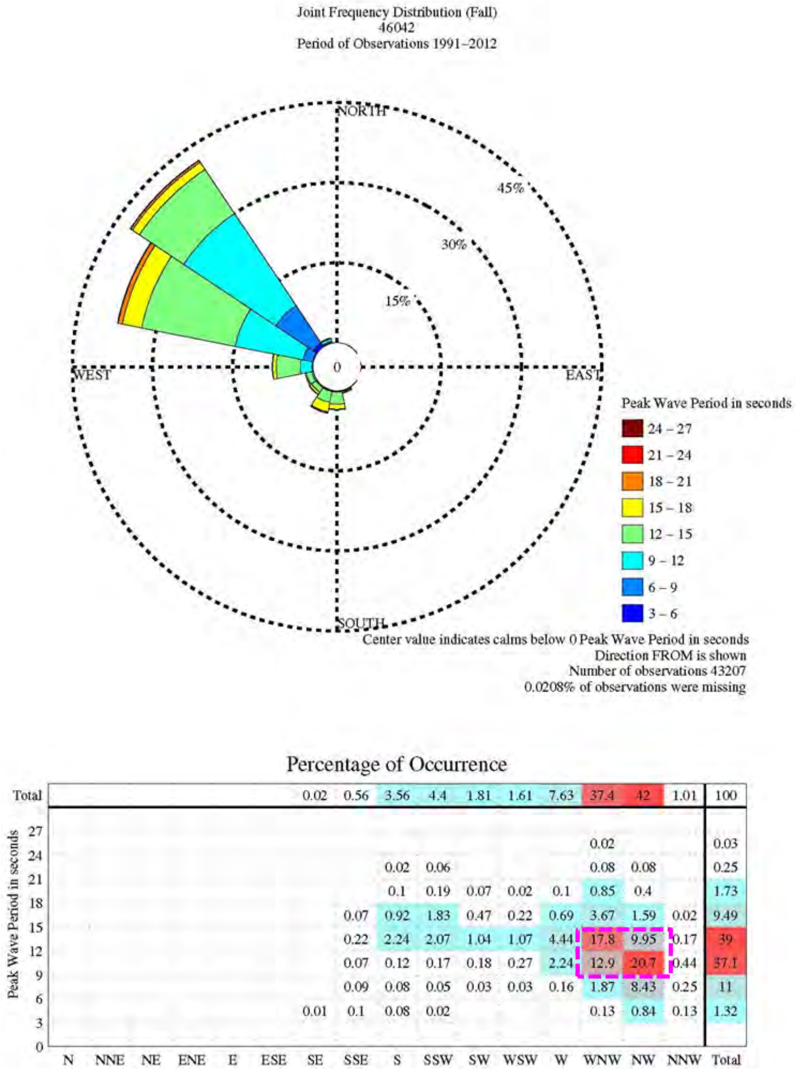
Joint Frequency Distribution (Fall)
46042
Period of Observations 1991–2012

Center value indicates calms below 0 Significant Wave Height in meters
Direction FROM is shown
Number of observations 43207
0.0208% of observations were missing

Percentage of Occurrence

	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total							0.02	0.56	3.56	4.4	1.81	1.61	7.63	37.4	42	1.01	100
9 – 10.5																	
7.5 – 9													0.01				0.02
6 – 7.5													0.02	0.1	0.07		0.19
4.5 – 6													0.05	0.6	0.79	0.01	1.47
3 – 4.5								0.05	0.05	0.06	0.05	0.01	0.57	5.19	6.75	0.09	12.8
1.5 – 3								0.25	1.07	1.28	0.51	0.57	4.06	24.4	28	0.67	60.8
0 – 1.5							0.01	0.25	2.44	3.05	1.25	1.02	2.91	7.12	6.38	0.23	24.7

Figure 10-25: Fall Wave Height Rose at the Buoy 46042



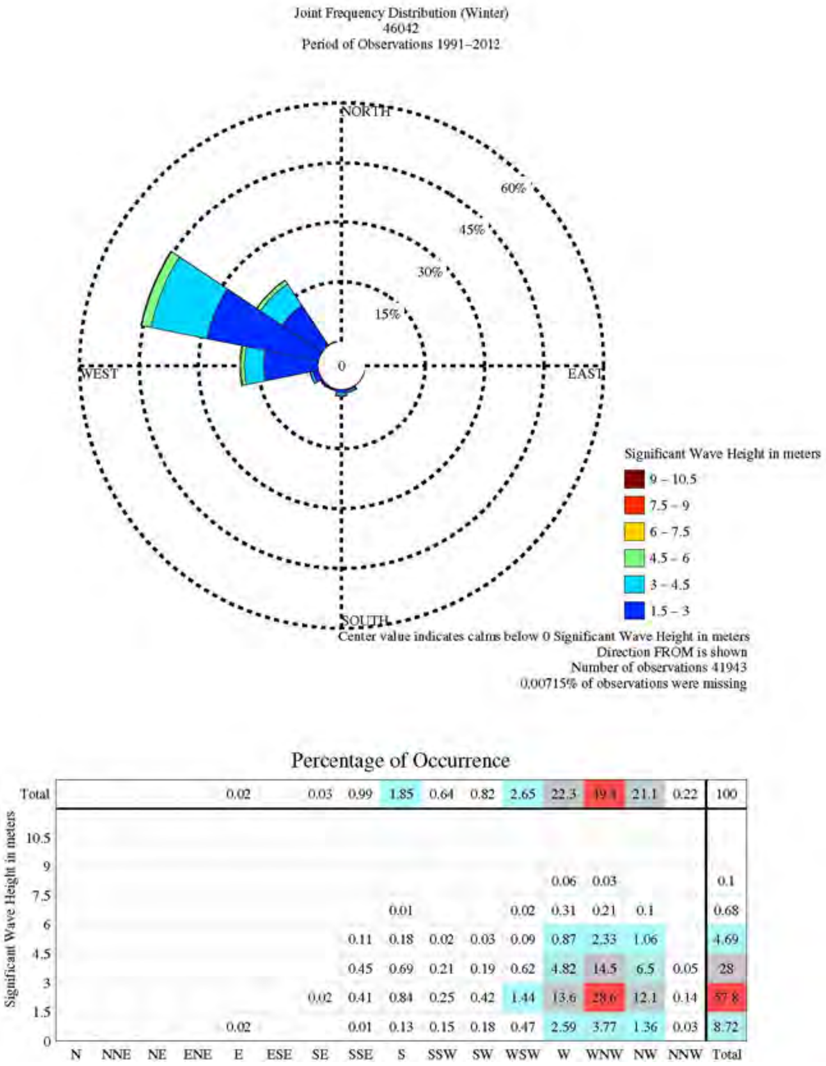
Joint Frequency Distribution (Fall)
46042
Period of Observations 1991–2012

Center value indicates calms below 0 Peak Wave Period in seconds
Direction FROM is shown
Number of observations 43207
0.0208% of observations were missing

Percentage of Occurrence

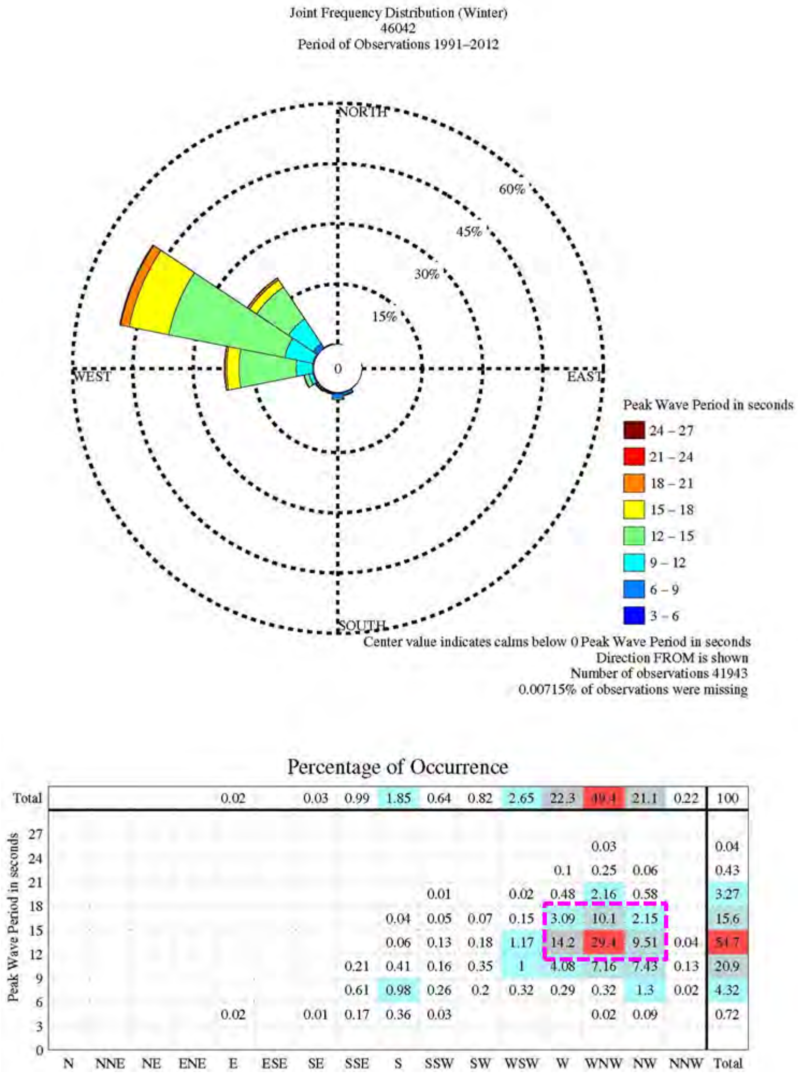
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total							0.02	0.56	3.56	4.4	1.81	1.61	7.63	37.4	42	1.01	100
24 – 27														0.02			0.03
21 – 24									0.02	0.06				0.08	0.08		0.25
18 – 21									0.1	0.19	0.07	0.02	0.1	0.85	0.4		1.73
15 – 18								0.07	0.92	1.83	0.47	0.22	0.69	3.67	1.59	0.02	9.49
12 – 15								0.22	2.24	2.07	1.04	1.07	4.44	17.8	9.95	0.17	39
9 – 12								0.07	0.12	0.17	0.18	0.27	2.24	12.9	20.7	0.44	37.1
6 – 9								0.09	0.08	0.05	0.03	0.03	0.16	1.87	8.43	0.25	11
3 – 6							0.01	0.1	0.08	0.02				0.13	0.84	0.13	1.32

Figure 10-26: Fall Wave Period Rose at the Buoy 46042



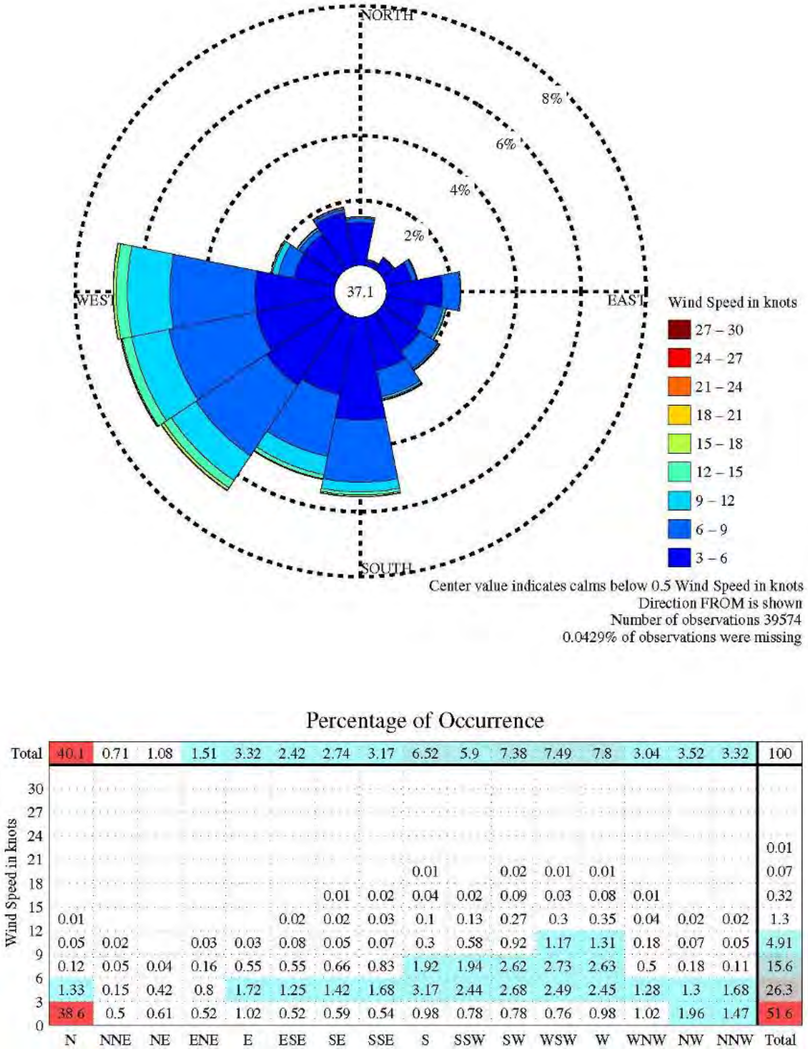
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total					0.02		0.03	0.99	1.85	0.64	0.82	2.65	22.3	49.4	21.1	0.22	100
													0.06	0.03			0.1
									0.01			0.02	0.31	0.21	0.1		0.68
								0.11	0.18	0.02	0.03	0.09	0.87	2.33	1.06		4.69
								0.45	0.69	0.21	0.19	0.62	4.82	14.5	6.5	0.05	28
							0.02	0.41	0.84	0.25	0.42	1.44	13.6	28.6	12.1	0.14	57.8
					0.02			0.01	0.13	0.15	0.18	0.47	2.59	3.77	1.36	0.03	8.72

Figure 10-27: Winter Wave Height Rose at the Buoy 46042



	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total					0.02		0.03	0.99	1.85	0.64	0.82	2.65	22.3	49.4	21.1	0.22	100
														0.03			0.04
													0.1	0.25	0.06		0.43
										0.01		0.02	0.48	2.16	0.58		3.27
									0.04	0.05	0.07	0.15	3.09	10.1	2.15		15.6
									0.06	0.13	0.18	1.17	14.2	29.4	9.51	0.04	54.7
								0.21	0.41	0.16	0.35	1	4.08	7.16	7.43	0.13	20.9
								0.61	0.98	0.26	0.2	0.32	0.29	0.32	1.3	0.02	4.32
					0.02		0.01	0.17	0.36	0.03				0.02	0.09		0.72

Figure 10-28: Winter Wave Period Rose at the Buoy 46042

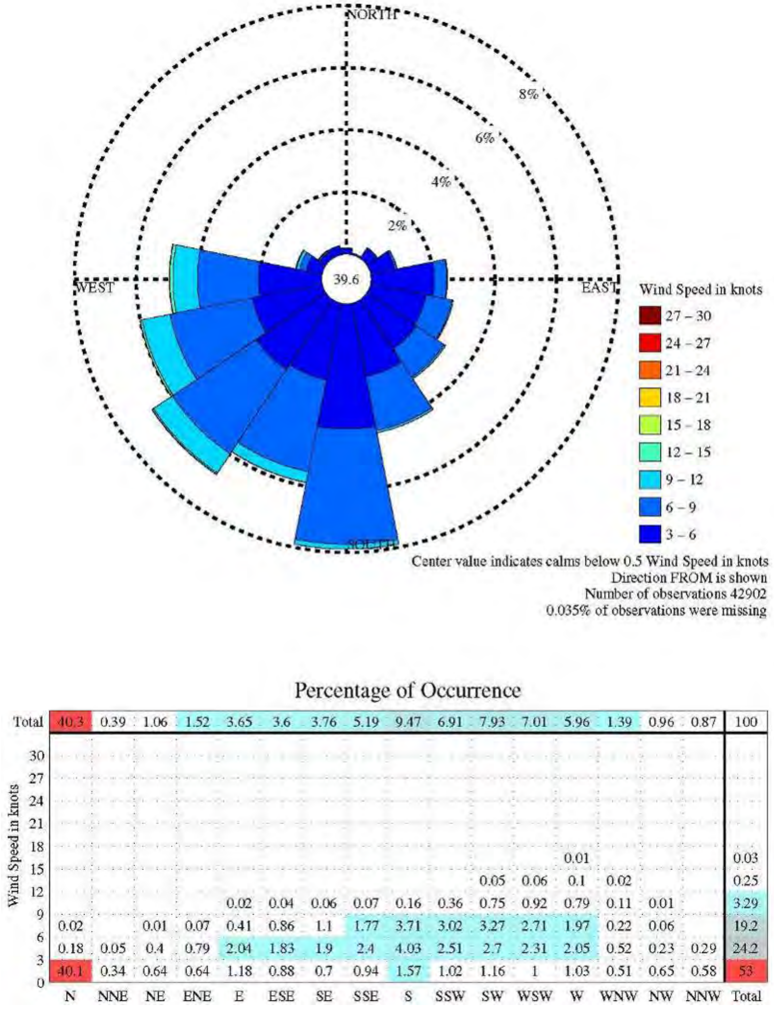


Center value indicates calms below 0.5 Wind Speed in knots
Direction FROM is shown
Number of observations 39574
0.0429% of observations were missing

Percentage of Occurrence

Wind Speed in knots	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	40.1	0.71	1.08	1.51	3.32	2.42	2.74	3.17	6.52	5.9	7.38	7.49	7.8	3.04	3.52	3.32	100
27 – 30																	
24 – 27																	
21 – 24																	0.01
18 – 21									0.01		0.02	0.01	0.01				0.07
15 – 18							0.01	0.02	0.04	0.02	0.09	0.03	0.08	0.01			0.32
12 – 15	0.01					0.02	0.02	0.03	0.1	0.13	0.27	0.3	0.35	0.04	0.02	0.02	1.3
9 – 12	0.05	0.02		0.03	0.03	0.08	0.05	0.07	0.3	0.58	0.92	1.17	1.31	0.18	0.07	0.05	4.91
6 – 9	0.12	0.05	0.04	0.16	0.55	0.55	0.66	0.83	1.92	1.94	2.62	2.73	2.63	0.5	0.18	0.11	15.6
3 – 6	1.33	0.15	0.42	0.8	1.72	1.25	1.42	1.68	3.17	2.44	2.68	2.49	2.45	1.28	1.3	1.68	26.3
0 – 3	38.6	0.5	0.61	0.52	1.02	0.52	0.59	0.54	0.98	0.78	0.78	0.76	0.98	1.02	1.96	1.47	51.6

Figure 10-29: Spring Wind Rose at the Watsonville Airport

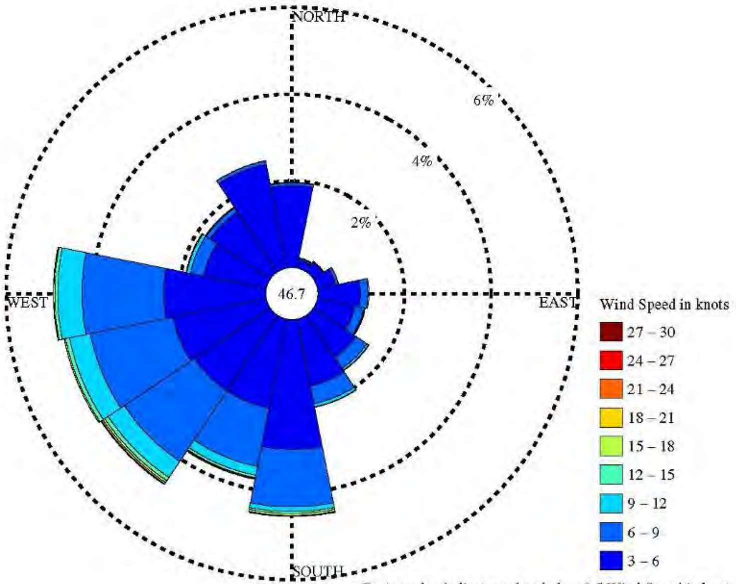


Center value indicates calms below 0.5 Wind Speed in knots
Direction FROM is shown
Number of observations 42902
0.035% of observations were missing

Percentage of Occurrence

Wind Speed in knots	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	40.3	0.39	1.06	1.52	3.65	3.6	3.76	5.19	9.47	6.91	7.93	7.01	5.96	1.39	0.96	0.87	100
27 – 30																	
24 – 27																	
21 – 24																	
18 – 21																	
15 – 18													0.01				0.03
12 – 15											0.05	0.06	0.1	0.02			0.25
9 – 12					0.02	0.04	0.06	0.07	0.16	0.36	0.75	0.92	0.79	0.11	0.01		3.29
6 – 9	0.02		0.01	0.07	0.41	0.86	1.1	1.77	3.71	3.02	3.27	2.71	1.97	0.22	0.06		19.2
3 – 6	0.18	0.05	0.4	0.79	2.04	1.83	1.9	2.4	4.03	2.51	2.7	2.31	2.05	0.52	0.23	0.29	24.2
0 – 3	40.1	0.34	0.64	0.64	1.18	0.88	0.7	0.94	1.57	1.02	1.16	1	1.03	0.51	0.65	0.58	53

Figure 10-30: Summer Wind Rose at the Watsonville Airport

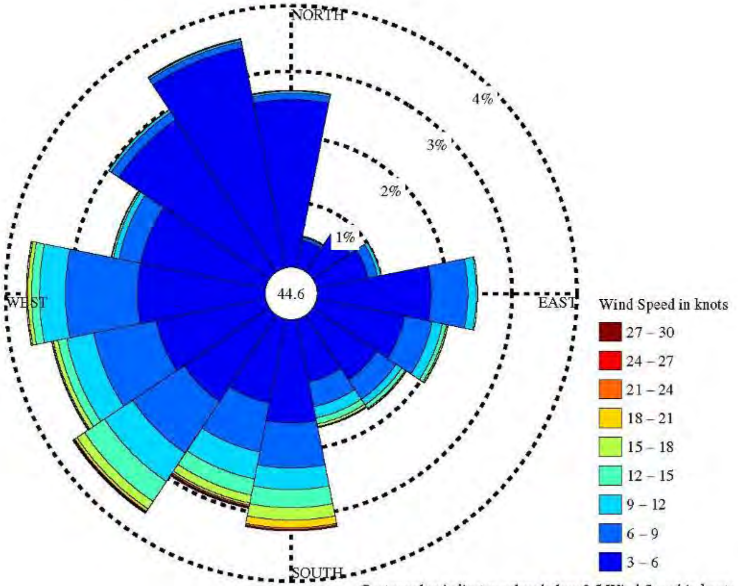


Center value indicates calms below 0.5 Wind Speed in knots
Direction FROM is shown
Number of observations 41509
0.0313% of observations were missing

Percentage of Occurrence

Wind Speed in knots	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	50.2	0.7	0.75	0.95	2.09	1.75	2.23	2.85	5.78	4.57	5.51	5.57	6.05	3.04	3.84	4.17	100
30																	
27																	
																	0.01
24																	
											0.01						0.03
21																	
									0.01	0.01	0.04						0.07
18																	
									0.05	0.01	0.05	0.03	0.02				0.17
15																	
						0.01			0.05	0.06	0.09	0.09	0.07	0.02			0.42
12																	
		0.01			0.02	0.02	0.05	0.07	0.11	0.21	0.45	0.49	0.56	0.08	0.02		2.12
9																	
	0.06	0.03	0.02	0.07	0.17	0.21	0.3	0.42	1.33	1.37	1.96	1.95	1.89	0.31	0.11	0.08	10.3
6																	
	1.89	0.19	0.27	0.42	0.98	0.88	1.18	1.57	2.99	2.1	2.1	2.2	2.36	1.48	1.66	2.44	24.7
3																	
	48.2	0.46	0.46	0.46	0.91	0.62	0.69	0.77	1.24	0.8	0.81	0.81	1.14	1.15	2.04	1.64	62.2
0																	

Figure 10-31: Fall Wind Rose at the Watsonville Airport



Center value indicates calms below 0.5 Wind Speed in knots
Direction FROM is shown
Number of observations 39616
0.0202% of observations were missing

Percentage of Occurrence

Wind Speed in knots	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	49.4	1.19	1.49	1.61	3.46	2.63	2.28	2.2	3.96	3.45	4.23	3.95	4.79	3.93	5.74	5.69	100
30																	
										0.02							0.03
27																	
									0.02	0.01	0.01						0.04
24																	
									0.03	0.02	0.02						0.08
21																	
								0.01	0.12	0.03	0.04	0.02	0.02				0.25
18																	
						0.02	0.02	0.04	0.19	0.12	0.14	0.07	0.06	0.01			0.68
15																	
	0.01				0.02	0.06	0.07	0.09	0.28	0.25	0.37	0.14	0.12	0.03		0.01	1.46
12																	
	0.03	0.01	0.02	0.05	0.13	0.16	0.1	0.18	0.33	0.42	0.52	0.43	0.4	0.09	0.05	0.03	2.94
9																	
	0.1	0.05	0.12	0.15	0.57	0.43	0.36	0.39	0.69	0.77	0.95	0.98	1.11	0.34	0.14	0.1	7.24
6																	
	2.57	0.43	0.69	0.8	1.75	1.38	1.19	0.98	1.58	1.31	1.58	1.72	1.97	1.96	2.79	3.43	26.1
3																	
	46.7	0.7	0.67	0.6	0.98	0.57	0.54	0.5	0.72	0.5	0.59	0.59	1.11	1.5	2.75	2.11	61.1
0																	

Figure 10-32: Winter Wind Rose at the Watsonville Airport

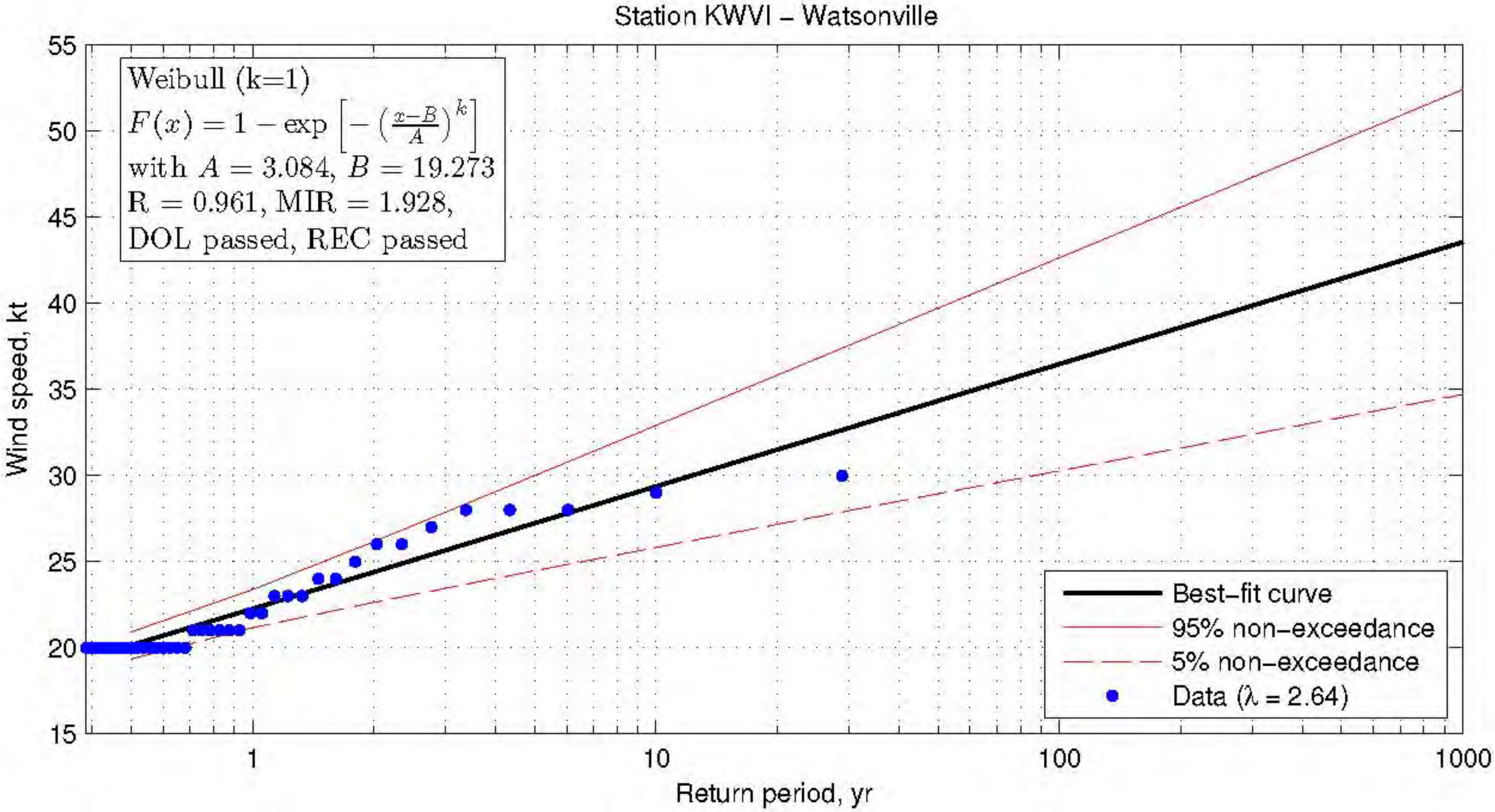


Figure 10-33: Extreme Winds Analysis for the Watsonville Airport